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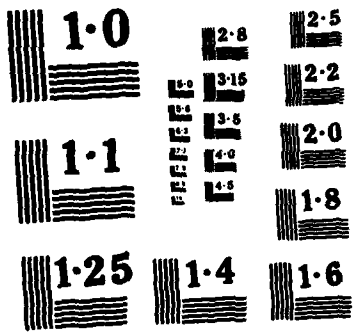

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AN EVALUATION OF THE USE OF PORE PRESSURE COEFFICIENTS IN THE ANALYSIS OF RAPID DRAWDOWN IN EARTH SLOPES

by

Richard Van Saun, B. S. C. E. , M. E. C. E

DISSERTATION

Presented to the Faculty of the Graduate School of

The University of Texas at Austin

in Partial Fulfillment

of the Requirements

for the Degree of

DOCTOR OF PHILOSOPHY

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IN EARTH SLOPES

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AN EVALUATION OF THE USE OF PORE PRESSURE COEFFICIENTS

IN THE ANALYSIS OF RAPID DRAWDOWN

IN EARTH SLOPES

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The use of pore water pressure coefficients in estimating the pore water pressure changes which are induced in an earth slope as the result of rapid drawdown is evaluated in this study. Equations which have been proposed in the literature which relate changes in pore pressure to principal changes in stress through empirically determined pore pressure coefficients serve as the basis for the evaluations conducted. A data bank of published soil shear test results was compiled and used to evaluate the influence of load path and testing variables on values of the pore pressure coefficient. An investigation was conducted to determine which, if either, of the the two forms of the pore pressure coefficient, Skempton's A or the pore pressure coefficient based on octahedral stresses, a, was independent of load

path such as triaxial compression or triaxial extension. In addition, the influence which plane strain has on the values of the pore pressure coefficient was investigated. The principal changes in stress which are induced in an earth slope as the result of rapid drawdown were estimated using the finite element method. Bishop (1954) proposed using pore pressure coefficients to estimate the changes in pore water pressure induced in an earth slope as the result of rapid drawdown. The assumptions which Bishop made in developing his method were evaluated using the principal changes in stress calculated in this study from the finite element method. In addition, values of the pore pressure coefficient which would be required to give the values of the change in pore water pressure obtained from Bishop's method were calculated using principal changes in stress estimated from the finite element method. Changes in pore pressure were calculated using measured values of the pore pressure coefficient. These values were compared with changes in pore pressure calculated using Bishop's procedure.

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LIST OF SYMBOLS

A	Skempton's pore pressure coefficient which expresses the change in pore water pressure caused by the application of a shear stress during undrained loading
$\bar{A}$	alternate form of the A coefficient = $A \cdot B$
AASHTO	American Association of State Highway and Transportation Officials
a	Pore pressure coefficient which expresses the change in pore water pressure caused by the application of a shear stress during undrained loading used in pore pressure equations which include the inter mediate principal change in stress.
$a\sqrt{2}$	alternate form of the a coefficient which is used for axially symmetric loading
$\bar{a}$	alternate form of the a or $a\sqrt{2}$ coefficient = $a \cdot B$
B	pore pressure coefficient which expresses the change in pore water pressure due to the application of an all-around confining stress
$\bar{B}$	overall pore pressure coefficient = $\Delta u / \Delta \sigma_1$
b	Intermediate Principal Stress Index
CCR	clay core - rockfill dam construction
CE	compactive effort
CMC	compaction moisture content
CS	consolidated slurry
D_v	normalized vertical depth
DWR	Department of Water Resources compaction criteria
E_s	soil modulus
H	Homogeneous earth dam construction

D_h	normalized horizontal distance from the face of the slope
HM	Harvard miniature compaction mold
HMK	kneading compaction in a Harvard miniature mold
HMS	static compaction in a Harvard miniature mold
h	vertical height
K_c	consolidation effective principal stress ratio
MA	modified AASHTO compaction criteria
MP	Modified Proctor compaction criteria
OCR	overconsolidation ratio
R	remolded sample
SP	Standard Proctor compaction crtiteria
TC	triaxial compression
TE	triaxial extension
UPS	undisturbed piston samples
UBS	undisturbed block samples
u	pore water pressure
Δu	change in pore water pressure
u_o	initial pore water pressure
x	horizontal distance
y	vertical distance
ϵ_f	strain at failure
γ_w	unit weight of water

$\Delta\sigma_1$	major principal change in stress
$\Delta\sigma_2$	intermediate principal change in stress
$\Delta\sigma_3$	minor principal change in stress
σ_c'	effective confining stress
σ_{oct}	octahedral normal stress
σ_{vc}'	vertical effective confining stress
σ_{vm}'	maximum vertical effective confining stress
σ_{vo}'	current vertical effective confining stress
τ_{oct}	octahedral shear stress

CHAPTER 1

INTRODUCTION

BACKGROUND

There are numerous cases in which the Geotechnical Engineer is required to evaluate the stability of submerged earth slopes. While the most obvious example is that of the upstream portion of an earth dam, submergence also influences the analysis of river and canal embankments, flood control levees, and the natural slopes within a reservoir containment area. In these cases, the most critical period in terms of stability typically occurs when the water level adjacent to the slope is lowered after steady state seepage conditions have been established. In fact, Sherard et al. (1963) report that all known slope failures in the upstream portion of in-service earth dams have occurred following a fall in the reservoir level. A sudden lowering of the reservoir level adjacent to an earth slope is commonly referred to as rapid drawdown. The term "rapid" is relative in this usage and refers to the relationship which exists between the rate (length/time) at which the water level drops and the rate at which the slope is able to drain. Therefore, a rapid drawdown condition exists when slope drainage lags behind reservoir lowering. When upstream slope failures have been induced by rapid drawdown, the rate of reservoir lowering was relatively low. In six cases of drawdown induced slope failures, the drawdown rate was between .3 and .5 ft/day

(Sherard, 1953). In all, a total of twenty upstream slope failures in earth dams due to rapid drawdown have been documented and these are listed in Table 1.1. In addition, Morgenstern (1963) reports that drawdown induced slides have occurred in the natural slopes of Franklin D. Roosevelt Lake. While upstream slope failures have never caused breaching of a dam (Sherard, 1963) their effects in terms of damage to outlet works, reduction in reservoir capacity and loss of dam service while repairs are being made can be significant. Therefore, it is critical to investigate the stability of earth slopes which may be subjected to rapid drawdown.

The procedures for evaluating the stability of an earth slope in common use today are based on the principles of limit equilibrium mechanics. During the past fifty years, a significant number of different procedures to evaluate the stability of an earth slope based on limit equilibrium have been developed. Fortunately, research in this area has been intense and the limitations and accuracy of the different procedures are documented (Wright et al., 1973). To apply these procedures, the shear strength of the soil along an assumed shear surface must be estimated. In this regard, the designer must choose between two essentially different methods of determining the shear strength of the soil: the total stress method and the effective stress method. However, as Lambe and Whitman (1969) explain, neither method holds an obvious advantage because "the same gaps in our knowledge which make it difficult to create the proper test conditions...to determine undrained strength for use in the total

Table 1.1 Summary of Upstream Slope Failures in Earth Dams Following Rapid Drawdown.

Dam Name	Type	Height (ft)	Upstream Slope	Downstream Slope	Rate (ft/day)	Reference
Aiai-ike	N/A	N/A	N/A	N/A	N/A	Akai (1958)
Bear Gulch	H	64	3:1	2:1	0.3	Sherard (1953)
Belle Fourche	H	115	2:1	2:1	0.3	Sherard (1953)
Brush Hollow	H	73	3:1	2:1	0.2	Sherard (1953)
Cercey	H	38	2.4:1	2:1	N/A	Mayer (1936)
Charmes	H	55	1.5:1	1.9:1	N/A	Mayer (1936)
Eildon	CR	140	N/A	N/A	N/A	Schatz (1936)
Forsyth	H	65	2:1	1.5:1	15	Sherard (1953)
Fruitgrowers	H	36	3:1	2:1	25	Sherard (1953)
Groisbois	H	58	2:1	2.5:1	N/A	Mayer (1936)
Mount Pisgah	H	76	1.5:1	2:1	0.4	Sherard (1953)
Narraquinnep	H	76	3:1	2:1	0.3	Babb (1968)
Palakmati	H	46	2-3:1	2-3:1	N/A	Rao (1961)
Pilcarcitos	H	95	2.5:1	N/A	1.8	ICOLD (1962)
Siburua	H	50	2.5:1	2:1	N/A	Wolfskill (66)
Standley Lake	H	113	2:1	2:1	0.5	Sherard (1953)
Tittesworth	N/A	N/A	N/A	N/A	N/A	Twort (1964)
Utica	N/A	N/A	N/A	N/A	N/A	Reinius (1948)
Wassey	H	54	1.5:1	1.9:1	1.1	Mayer (1936)
Willingdon	H	55	2:1	3:1	N/A	Rao (1961)

(H - Homogeneous, CR - Clay Core,Rockfill, N/A - Data not provided)

stress method...make it difficult to predict pore pressure changes" which are required if the effective stress method is to be used.The stability of an earth slope can be estimated using the effective stress method, provided that the pore pressures acting along an assumed failure surface are known. During rapid drawdown, the stresses acting at a point in the earth slope change in response to the removal of the water load from the face of the slope. For impervious soils, drawdown can be conservatively assumed to occur instantaneously under conditions of no volume change. In this case, the changes in total stress will cause changes in the pore water pressure. By relating changes in pore pressure to changes in total stress the final pore water pressure can be estimated and the effective stress method can be used for stability analysis.

PURPOSE AND SCOPE

The main objective of this study is to evaluate the use of pore pressure coefficients to estimate the pore water pressures which are induced in an earth slope as the result of rapid drawdown. A number of researchers (Skempton (1954), Bishop (1954), Henkel (1960), and Perloff and Baron (1976)) have proposed relating the changes in pore water pressure which occur during undrained shear of soil to principal changes in stress through experimentally determined pore pressure coefficients. The forms of the pore pressure coefficient equation which have been proposed in the literature are reviewed in

Chapter Two to identify their differences and similarities and to establish definitions of the terms used in these equations for the purposes of this study. An extensive data bank of published test results was compiled and an investigation was conducted, as summarized in Chapter Three, to determine which, if any, of the different forms of the pore pressure coefficient is independent of loading path (such as triaxial compression versus triaxial extension) and to determine how testing variables might influence that relationship. Finite element computations were performed to investigate the potential magnitude and pattern of the principal changes in stress which are induced in an earth slope when it is subjected to rapid drawdown. The computer code used, assumptions made, and the findings of these analyses are presented in Chapter Four. A method was proposed by Bishop (1954) which uses the pore pressure coefficient equation to estimate the pore water pressures in an earth dam following rapid drawdown. The results of an investigation conducted to evaluate the assumptions made by Bishop are presented in Chapter Five. Changes in pore pressure were computed using measured values of the pore pressure coefficient and stresses from the finite element method. These changes in pore pressure were compared with changes in pore pressure calculated using Bishop's procedure as reported in Chapter Six.

CHAPTER 2

PORE PRESSURE COEFFICIENT EQUATIONS

INTRODUCTION

Several theoretical expressions have been proposed which relate changes in pore pressure to principal changes in total stress during undrained shear. All of these expressions establish this relationship through experimentally determined pore pressure coefficients. A review of the different forms of these expressions, which have been proposed in the literature, is presented in this chapter.

SKEMPTON'S PORE PRESSURE COEFFICIENTS

In cases of undrained shear of soils, the pore pressure changes, Δu, can be related to the principal changes in stress, $\Delta\sigma_1$ and $\Delta\sigma_3$, by the equation:

$$\Delta u = B\left[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\right] \qquad 2.1$$

which was proposed by Skempton (1954), where A and B are experimentally determined "pore pressure coefficients". As shown in Figure 2.1, Skempton considered that the application of the stresses $\Delta\sigma_1$ and $\Delta\sigma_3$ takes place in two distinct phases. First, the soil element is subjected to an all around confining stress, $\Delta\sigma_3$, which will produce a corresponding pore pressure change, Δu_a. The magnitude of Δu_a is

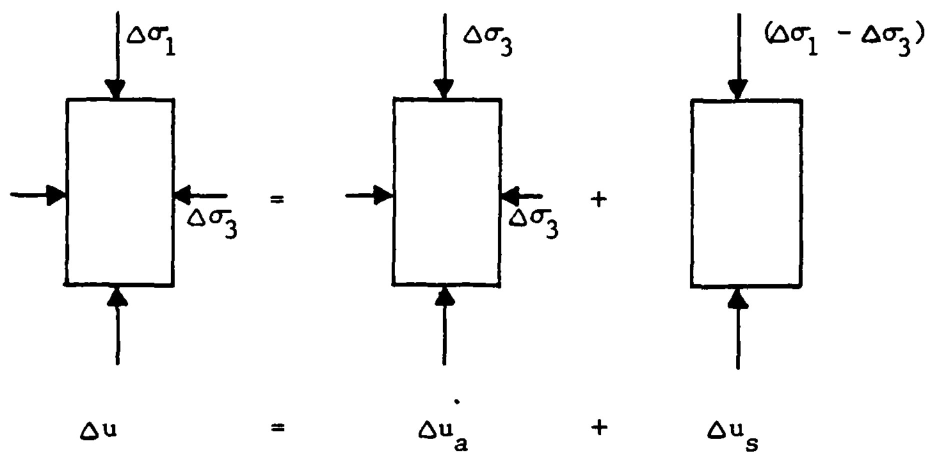


Fig 2.1 Equivalent Stress Block Diagram for Soil Specimen Subjected to Stresses $\Delta\sigma_1$ and $\Delta\sigma_3$ During Undrained Loading in Triaxial Compression.

related to the degree of saturation of the soil which is measured experimentally and expressed by the pore pressure coefficient B, such that:

$$\Delta u_a = B\,\Delta\sigma_3 \qquad 2.2$$

Theoretically, values of the B coefficient range from 0 for completely dry soils to nearly 1 for completely saturated soils. However, Lee et al. (1969) report that the value of B at 100 percent saturation is dependent upon the relation of the compressibility of the pore fluid to the compressibility of the soil structure. At 100 percent saturation, their test data show that the value of the B coefficient can range from .85 to 1.00.

During the second phase of loading considered by Skempton, the application of a shear stress, $(\Delta\sigma_1 - \Delta\sigma_3)$, produces a corresponding pore pressure change, Δu_s. The magnitude of Δu_s is related to the degree of saturation of the soil which is reflected in the pore pressure coefficient, B. It is also related to the shear induced volume change tendencies of the soil which are expressed in the pore pressure coefficient, A, such that:

$$\Delta u_s = B \cdot A\,(\Delta\sigma_1 - \Delta\sigma_3) \qquad 2.3$$

For soils such as loose sands and normally consolidated clays, the soil tends to compress during shear, and the pore pressure coefficient, A, will have a positive value. Conversely, the value of A will be negative if the soil tends to dilate during shear.

The total pore pressure change induced by the two phases of loading is:

$$\Delta u = \Delta u_a + \Delta u_s \qquad 2.4$$

Substituting Equations 2.2 and 2.3 into Equation 2.4 gives:

$$\Delta u = B \Delta\sigma_3 + B \cdot A\ (\Delta\sigma_1 - \Delta\sigma_3) \qquad 2.5$$

A useful form of Equation 2.5 is obtained by defining an additional pore pressure coefficient $\bar{A}$, such that:

$$\bar{A} = B \cdot A \qquad 2.6$$

giving:

$$\Delta u = B \Delta\sigma_3 + \bar{A}\ (\Delta\sigma_1 - \Delta\sigma_3) \qquad 2.7$$

For saturated soils where the value of B is equal to one, A is equal to $\bar{A}$ and Equation 2.7 can be simplified to:

$$\Delta u = \Delta\sigma_3 + A\ (\Delta\sigma_1 - \Delta\sigma_3) \qquad 2.8$$

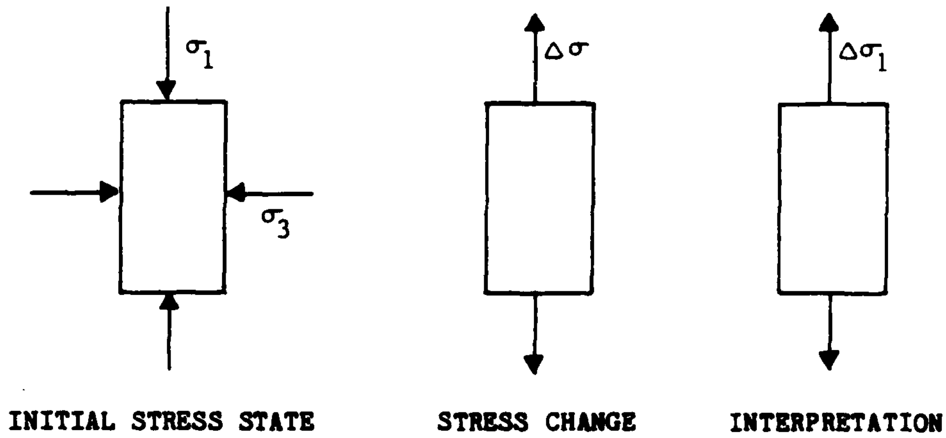
Because the coefficients B, A, and $\bar{A}$ were first introduced by Skempton, they are commonly referred to as "Skempton's pore pressure coefficients."

DEFINITION OF $\Delta\sigma_1$ and $\Delta\sigma_3$

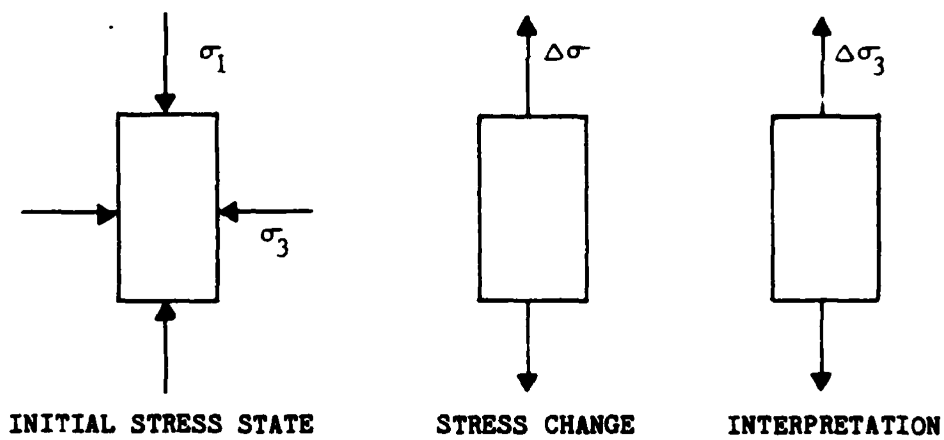
The literature contains two different interpretations of the changes in stress $\Delta\sigma_1$ and $\Delta\sigma_3$ when they are used in Skempton's pore pressure equation. In one interpretation, $\Delta\sigma_1$ and $\Delta\sigma_3$ are defined to be the changes in the major and minor principal stresses, respectively. This interpretation implies that the definition of the

changes in stress is related to the geometric orientation of the major and the minor principal stresses prior to shear. It is apparent that this was Skempton's (1960) interpretation when he stated that " $\Delta\sigma_1$ may be greater than or less than $\Delta\sigma_3$". Under the second interpretation, as proposed by Law and Holtz (1978), the definition of the changes in stress is considered to be independent of the initial stress state. The algebraically largest change in stress is defined to be the major principal change in stress, $\Delta\sigma_1$. The algebraically smallest change in stress is defined to be the minor principal change in stress, $\Delta\sigma_3$.

When the principal stresses are reoriented during shear, the differences in these two interpretations will lead to considerable differences in the expression for A obtained from Skempton's equation (Eq. 2.8). To examine the impact of these two different interpretations of the change in stress on the expression for the A coefficient, we will consider the case of an undrained triaxial shear test conducted on an anisotropically consolidated specimen in which the axial stress is decreased to failure while the cell pressure is held constant. Stress block analogies are presented in Figure 2.2 to aid in the explanation of how the two interpretations of the changes in stress would be applied to this test. The initial stress state shown in Figure 2.2 reflects anisotropic consolidation in which the axial stress σ_1 is greater than the lateral stress σ_3. The change in stress, $\Delta\sigma$, reflects undrained shear of the soil specimen in which the axial stress is decreased to failure while the lateral stress is held



a) Interpretation I - Change in Principal Stress



b) Interpretation II - Principal Change in Stress

Fig 2.2 Possible Interpretations of the Definition of the Changes in Stress $\Delta\sigma_1$ and $\Delta\sigma_3$

constant. The subscript to be applied to $\Delta\sigma$ (1, 2, or 3) is dependent upon the interpretation used to define the change in stress.

Under the first interpretation, the definition of the change in stress $\Delta\sigma$ is dependent upon the geometric orientation of the principal stress prior to shear. As shown in Figure 2.2a, the change in stress $\Delta\sigma$ was applied in the direction of the initial major principal stress σ_1 and under this interpretation would be defined as the change in the major principal stress, $\Delta\sigma_1$. The lateral stress is held constant during this test and is in the direction of the initial minor principal stress. Therefore, it is defined to be the minor principal change in stress and has a value of zero. Substituting these values into Skempton's equation (Eq. 2.8) gives:

$$\Delta u = A\,\Delta\sigma_1 \qquad 2.9$$

which, rearranged gives:

$$A = \Delta u / \Delta\sigma_1 \qquad 2.10$$

Under the second interpretation, the definition of the change in stress is independent of the initial stress state. As shown in Figure 2.2b, the change in stress, $\Delta\sigma$, is tensile. Since standard geotechnical convention defines compressive loading to be positive and because the lateral change in stress is zero, under this interpretation, the change in stress, $\Delta\sigma$, would be defined as the minor principal change in stress $\Delta\sigma_3$ because it is the algebraically smallest change in stress. Similarly, the lateral change in stress, which is equal to zero, would be defined as the major principal change in stress, $\Delta\sigma_1$, because it is the algebraically largest change in

stress. Substituting these values into Skempton's equation gives:

$$\Delta u = \Delta\sigma_3 - A\ \Delta\sigma_3 \qquad 2.11$$

which, rearranged gives:

$$A = 1 - \Delta u / \Delta\sigma_3 \qquad 2.12$$

A comparison of Equations 2.10 and 2.12 shows that the different interpretations of the changes in stress result in significantly different expressions for the A coefficient. It is important to note that these differences in the expression for A are not caused by a difference in soil behavior but are due solely to the different interpretations of the change in stress used. For purposes of this study, the preferred interpretation of the change in stress is the one which gives the most consistent values of the pore pressure coefficient, A.

To determine which of the two interpretations provide more consistent results, both definitions of the changes in stress were used to calculate values of the A coefficient at failure (A_f) for a series of triaxial shear tests reported by Paster (1982). All of the tests were performed on specimens prepared in the laboratory from a commercially available kaolinite. The test series included triaxial compression and triaxial extension tests conducted on both isotropically and anisotropically consolidated specimens. The compression tests were performed by increasing the axial stress to failure while holding the cell pressure constant. In the extension tests, the cell pressure was held constant and the axial stress was decreased to failure. Values of the pore pressure coefficient, A_f,

calculated from the test results reported by Paster which were obtained by applying the two different interpretations of the change in stress are summarized in Table 2.1. A complete summary of Paster's test data is contained in Appendix B. The test type is designated as CIUC to indicate Consolidated Isotropically, Undrained axial Compression or CAUE to indicate Consolidated Anisotropically, Undrained axial Extension. The CIUC, CIUE, and CAUC tests did not involve reorientation of the directions of the principal stresses during shear. And, as can be seen in Table 2.1, the two different interpretations of the change in stress produce identical calculated values of the pore pressure coefficient. However, a 90 degree reorientation in the directions of the principal stresses occurs during an axial extension test conducted on an anisotropically consolidated specimen (CAUE). In this case, as can be seen in Table 2.1, the two different interpretations of the change is stress result in significantly different calculated values of the pore pressure coefficient. It is evident from Table 2.1 that, as suggested by Duncan and Seed (1966a) and Law and Holtz (1978), defining the changes in stress to be principal changes in stress rather than changes in principal stress produces more consistent values of the pore pressure coefficient, A_f. Because of these findings, changes in stress will be defined as principal changes in stress for purposes of this study where $\Delta\sigma_1$ is the algebraically largest and hence major principal change in stress and $\Delta\sigma_3$ is the algebraically smallest and, hence, minor principal change in stress.

Table 2.1 Summary of A_f Values for Kaolinite Calculated from the Results of Triaxial Compression and Extension Tests.

	INTERPRETATION OF A_f*	
TEST TYPE	CHANGE IN PRINCIPAL STRESS	PRINCIPAL CHANGE IN STRESS
CIUC-1	1.13	1.13
CIUC-2	1.14	1.14
CIUC-3	1.20	1.20
CIUC-4	1.28	1.28
CIUE-1	1.44	1.44
CIUE-2	1.45	1.45
CAUC-1	.68	.68
CAUC-2	.88	.88
CAUE-1	-.12	1.12
CAUE-2	-.17	1.17
CAUE-3	-.15	1.15

* Determined from Equation 2.8

PORE PRESSURE COEFFICIENT EQUATIONS WHICH INCORPORATE THE INTERMEDIATE PRINCIPAL CHANGE IN STRESS

Originally, Skempton (1954) expressed the change in pore water pressure, Δu, in a form which implied that it was independent of the intermediate principal change in stress, $\Delta\sigma_2$. Later, Henkel (1960) and Skempton (1960) proposed incorporating $\Delta\sigma_2$ into the equation. The equation which they introduced applied specifically to saturated soils in the undrained condition and took the form:

$$\Delta u = \frac{\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3}{3} + a\sqrt{(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2} \qquad 2.13$$

in which "a" is an empirical pore pressure coefficient. Subsequently, Henkel and Wade (1966) proposed expressing Equation 2.13 in terms of octahedral stresses. The octahedral normal stress, σ_{oct}, is defined as:

$$\sigma_{oct} = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3} \qquad 2.14$$

and the octahedral shear stress, τ_{oct}, is defined as:

$$\tau_{oct} = 1/3\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} \qquad 2.15$$

Introducing the definitions of the octahedral stresses into Equation 2.13 gives:

$$\Delta u = \Delta\sigma_{oct} + 3\,a\Delta(\tau_{oct}) \qquad 2.16$$

which in expanded form becomes:

$$\Delta u = \frac{\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3}{3} + a\,\Delta\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2} \qquad 2.17$$

A comparison of Equations 2.13 and 2.17 shows that they are not numerically equivalent because of the differences in the mathematical definition of $\Delta\tau_{oct}$. Equation 2.13 is expressed in a form which implies that:

$$\Delta\tau_{oct} = 1/3\sqrt{(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2} \qquad 2.18$$

where $\Delta\tau_{oct}$ is defined to be the octahedral shear component of the principal changes in stress and $\Delta\sigma_1$, $\Delta\sigma_2$, and $\Delta\sigma_3$ are the major, intermediate, and minor principal changes in stress, respectively. In contrast, Henkel and Wade (1966) and Perloff and Baron (1976) state that a more logical form of the pore pressure equation expresses $\Delta\tau_{oct}$ as the difference in the final and initial octahedral shear stresses, where:

$$\Delta\tau_{oct} = \tau_{oct_{final}} - \tau_{oct_{initial}} \qquad 2.19$$

For purposes of this study, the preferred form of the pore pressure equation based on octahedral stresses is the one which produces consistent values of the pore pressure coefficient, a. To determine this, both interpretations of $\Delta\tau_{oct}$ were used in the pore pressure equation to calculate the values of the pore pressure coefficient at failure, a_f, for a series of triaxial shear tests presented by Paster (1981). The details of this test series were reported in the previous section of this chapter. The calculated values of the pore pressure

coefficient, a_f, are summarized in Table 2.2. As can be seen, in tests where the major principal stresses are not reoriented during shear (CIUC, CIUE, CAUC), values of the pore pressure coefficient at failure, a_f, are not influenced by the definition of $\Delta\tau_{oct}$. However, when the principal stresses are reoriented during shear in the CAUE tests, more consistent values of the pore pressure coefficient are obtained when $\Delta\tau_{oct}$ is defined to be the octahedral shear component of the principal changes in stress. Therefore, this interpretation has been adopted for use in this study.

Perloff and Baron (1976) generalized Equation 2.13 to permit the analysis of partially saturated soils. Their derivation paralleled Skempton's (1954) original derivation but it was developed in terms of the octahedral stresses. The form of their equation is:

$$\Delta u = B\,\Delta\sigma_{oct} + 3\,B\,a\Delta\tau_{oct} \qquad 2.20$$

in which B is Skempton's original pore pressure coefficient. As with Skempton's equation a useful form of Equation 2.20 is obtained by defining an additional pore pressure coefficient $\bar{a}$ = Bxa giving:

$$\Delta u = B\,\Delta\sigma_{oct} + 3\,\bar{a}\Delta\tau_{oct} \qquad 2.21$$

In cases where B is equal to one 1, Equation 2.21 can be further simplified to:

$$\Delta u = \Delta\sigma_{oct} + 3a\Delta\tau_{oct} \qquad 2.22$$

Equations 2.20 through 2.22 will be referred to as the pore pressure equations based on octahedral stresses and the parameters "a" and "$\bar{a}$" as the pore pressure coefficients based on octahedral stresses. It should be noted that in using Perloff and Barons (1976) equations,

Table 2.2 Summary of a_f Values for Kaolinite Calculated from the Results of Triaxial Compression and Extension Tests.

TEST TYPE	INTERPRETATION OF a_f: CHANGE IN OCTAHEDRAL STRESS[1]	INTERPRETATION OF a_f: OCTAHEDRAL SHEAR COMPONENT OF THE CHANGE IN STRESS[2]
CIUC-1	.80	.80
CIUC-2	.81	.81
CIUC-3	.87	.87
CIUC-4	.95	.95
CIUE-1	.78	.78
CIUE-2	.78	.78
CAUC-1	.24	.24
CAUC-2	.39	.39
CAUE-1	-4.82	.32
CAUE-2	-8.39	.35
CAUE-3	-3.90	.34

1. Determined from Equation 2.17

2. Determined from Equation 2.13

their definition of $\Delta\tau_{oct}$ will not be used. Rather, $\Delta\tau_{oct}$ will be defined to be the octahedral shear component of the principal changes in stress for purposes of this study.

A COMPARISON OF THE PORE PRESSURE COEFFICIENTS FOR SHEAR

The pore pressure coefficients, A and a, can be related to one another for laboratory shear tests in which the loading is axially symmetric. Such relationships are presented and discussed in this section for the case of triaxial compression and triaxial extension.

Several paths can be followed to load a soil specimen in compression in the triaxial cell. The axial stress can be increased while the cell pressure is held constant (axial compression). The cell pressure can be decreased while the axial stress is held constant (lateral extension). Or, a combination of increasing axial stress with decreasing lateral stress can be used. Irrespective of the load path followed, the change in the axial stress is the major principal change in stress, $\Delta\sigma_1$, the change in the lateral stress is the minor principal change in stress, $\Delta\sigma_3$ and the intermediate principal change in stress, $\Delta\sigma_2$, is equal to the minor principal change in stress, $\Delta\sigma_3$. Substituting these values into Skempton's pore pressure equation (Eq. 2.8) gives:

$$\Delta u = \Delta\sigma_3 + A\,(\,\Delta\sigma_1 - \Delta\sigma_3\,) \qquad 2.23$$

and into the pore pressure equation based on octahedral stresses (Eq. 2.13) gives:

$$\Delta u = \frac{\Delta\sigma_1 + 2\cdot\Delta\sigma_3}{3} + a\sqrt{2}\,(\Delta\sigma_1 - \Delta\sigma_3) \qquad 2.24$$

Combining equation 2.23 and 2.24 and rearranging gives:

$$a\sqrt{2} = A - 1/3 \qquad 2.25$$

which provides a simple mathematical expression relating a and A for triaxial compression.

Several paths can be followed to load a soil specimen in extension in the triaxial cell. The axial stress can be decreased while the cell pressure is held constant (axial extension). The cell pressure can be increased while the axial stress is held constant (lateral compression). Or, a combination of decreasing axial stress with increasing lateral stress can be used. Irrespective of the load path followed, the change in the lateral stress is the major principal change in stress, $\Delta\sigma_1$, the change in the axial stress is the minor principal change in stress, $\Delta\sigma_3$ and the intermediate principal change in stress, $\Delta\sigma_2$, is equal to the major principal change in stress, $\Delta\sigma_1$. Substituting these values into Skempton's pore pressure equation (Eq. 2.8) gives:

$$\Delta u = \Delta\sigma_3 + A\,(\Delta\sigma_1 - \Delta\sigma_3) \qquad 2.26$$

and into the pore pressure equation based on octahedral stresses (Eq. 2.13) gives:

$$\Delta u = \frac{2\cdot\Delta\sigma_1 + \Delta\sigma_3}{3} + a\sqrt{2}\,(\Delta\sigma_1 - \Delta\sigma_3) \qquad 2.27$$

Combining equation 2.26 and 2.27 and rearranging gives:

$$a\sqrt{2} = A - 2/3 \qquad 2.28$$

which provides a simple mathematical expression relating a and A for triaxial extension.

While the pure form of the pore pressure coefficient based on octahedral stresses is "a", it is more convenient to express this pore pressure coefficient in the form $a\sqrt{2}$ for axially symmetric loading. This permits establishment of a simple numerical expression relating the two pore pressure coefficients. The pore pressure coefficient based on octahedral stresses expressed in the form $a\sqrt{2}$ will be used for the remainder of this study.

CONCLUSIONS

Two different forms of the pore pressure coefficient equation have been proposed in the literature. These equations incorporate two different pore pressure coefficients; Skempton's pore pressure coefficient, A, and the pore pressure coefficient based on octahedral stresses, a. An investigation to determine which form of the pore pressure coefficient is most independent of loading path, such as triaxial compression or triaxial extension, is presented in the following chapter.

CHAPTER 3

EVALUATION OF THE PORE PRESSURE COEFFICIENTS USING MEASURED SHEAR TEST DATA

Two forms of the pore pressure coefficient equation, Skempton's pore pressure equation and the pore pressure equation based on octahedral stresses, were reviewed in the previous chapter. The preferred form of these pore pressure equations is the one which produces values of the pore pressure coefficient which are relatively independent of the loading path used, such as triaxial compression and triaxial extension. Available experimental data were examined to determine which, if either, of the two forms of the pore pressure coefficient are unique or if one is more unique than the other irrespective of the loading path used. In addition, factors which might influence the degree of uniqueness were examined.

DATA COLLECTION AND REDUCTION

An extensive literature search was conducted to locate sources of data for use in this study. In order to permit direct comparison of the pore pressure coefficients, acceptable data were limited to cases of undrained shear testing for which pore pressure changes were reported and the only test variable was load path. Therefore, other

variables such as soil type, specimen preparation, initial effective principal stress ratio, and overconsolidation ratio were required to be identical for all loading paths. The majority of the test data which met these criteria were based on the results of triaxial compression and triaxial extension tests.

The data selected for use in this study are summarized in Table 3.1. These data represent over 140 triaxial shear tests conducted on 18 different soil types. The specific values selected for inclusion in Table 3.1 are those which are used in the evaluations presented in this chapter. A symbol has been assigned to each data set to facilitate identification in the graphical presentations which follow. Values of the pore pressure coefficient based on octahedral stresses ($a_f \sqrt{2}$) were not reported by any of the original authors and hence, these values had to be computed as part of this study. In the majority of the cases, values of A were reported by the original authors. However, in several instances, values of A had to be calculated in this study based on data extracted from tabulated or graphical results presented by the original author. All of the values of the pore pressure coefficients used in this study are those at failure, A_f and $a_f \sqrt{2}$, where failure is defined as being at the point where the principal stress difference ($\Delta\sigma_1 - \Delta\sigma_3$) was a maximum. In all cases, the degree of saturation is assumed to be 100 percent and the value of the B coefficient is assumed to be equal to one. The original test data used to prepare Table 3.1 are summarized on individual data sheets in Appendix B.

Table 3.1 Summary of Pore Pressure Coefficient Comparison Data from Triaxial Compression and Triaxial Extension Tests.

SOIL	SYMBOL USED	SAMPLE PREP	K_c	OCR	σ_c' (psi)	A_f TC	A_f TE	$a_f\sqrt{2}$ TC	$a_f\sqrt{2}$ TE	REFERENCE
Hackensack Valley	◇	UPS	0.65	1.0	42.7	1.35	1.07	1.02	0.40	Saxena (1974)
Drammen Clay	◇	UPS	1.00	4.0	14.2	0.11	0.71	-.22	0.04	NGI (1975)
Kars Clay	▽	UPS	0.75	1.0	10.6	0.34	0.73	0.01	0.06	Law (1978)
Gloucester Clay	▽	UPS	0.80	1.0	10.6	0.40	0.80	0.07	0.13	Law (1978)
Mexico City Clay	◇	UCS	1.00	1.0	7.1	0.69	0.98	0.35	0.32	Leon (1977)
"	◇	"	1.00	1.0	14.2	0.77	0.98	0.44	0.31	"
"	◇	"	1.00	1.0	28.4	0.95	0.96	0.61	0.29	"
Haney Clay	▽	UBS	1.00	1.0	85.3	.99	1.09	0.66	0.42	Vaid (1966)
Weathered Bangkok	□	U	1.00	2.0	5.0	0.59	0.99	0.26	0.33	Balasub.(1978)
Clay	□	"	1.00	1.7	6.0	0.75	1.10	0.42	0.43	"
"	□	"	1.00	1.0	15.0	1.07	0.85	0.73	0.18	"
"	□	"	1.00	1.0	30.0	1.11	0.95	0.78	0.29	"
"	□	"	1.00	1.0	40.0	1.14	1.03	0.81	0.36	"
"	□	"	1.00	1.0	60.0	1.14	1.10	0.81	0.43	"
Soft Bangkok Clay	□	U	1.00	1.0	20.0	0.83	1.05	0.50	0.38	"
"	□	"	1.00	1.0	30.0	1.06	1.05	0.72	0.38	"
"	□	"	1.00	1.0	40.0	1.12	1.22	0.79	0.55	"
"	□	"	1.00	1.0	50.0	0.98	0.87	0.65	0.20	"
Stiff Bangkok Clay	□	U	1.00	1.0	20.0	-.02	0.28	-.35	-.39	"
"	□	"	1.00	1.0	30.0	0.11	0.38	-.22	-.28	"
"	□	"	1.00	1.0	40.0	0.13	0.50	-.21	-.17	"
"	□	"	1.00	1.0	50.0	0.14	0.63	-.19	-.03	"
"	□	"	1.00	1.0	60.0	0.35	0.55	0.02	-.11	"
"	□	"	1.00	1.0	70.0	0.46	0.59	.013	-.08	"
"	□	"	1.00	1.0	80.0	0.40	0.55	0.07	-.12	"
"	□	"	1.00	1.0	90.0	0.27	0.68	-.06	0.01	"
Kaolinite (HY-UF)	▽	CS	1.00	1.0	29.8	1.21	1.45	0.87	0.78	Paster (1983)
"	▽	"	0.60	1.0	29.8	0.75	1.15	0.42	0.48	"

Table 3.1 (Continued)

SOIL	SYMBOL USED	SAMPLE PREP	K_c	OCR	σ_c' (psi)	A_f TC	A_f TE	$a_f\sqrt{2}$ TC	$a_f\sqrt{2}$ TE	REFERENCE
Boston Blue Clay	-▽-	CS	0.51	1.0	55.0	0.87	1.01	0.54	0.34	Ladd (1971)
"	-▽-	"	0.71	2.0	28.2	0.34	0.85	0.01	0.18	"
Gleason Clay	△	CS	1.00	1.0	30.0	1.23	1.37	0.90	0.70	Fennessey (1983)
"	△	"	1.00	1.0	30.0	1.18	1.38	0.86	0.71	"
"	△	"	0.57	1.0	30.4	1.13	0.96	0.80	0.30	"
"	△	"	0.66	1.0	30.0	1.23	1.01	0.90	0.35	"
"	△	"	0.77	1.0	30.2	1.55	1.13	1.22	0.46	"
Kaolinite (HY-UF)	△	"	0.60	1.0	30.2	0.76	1.18	0.44	0.51	"
Kaolin	-○-	CS	1.00	1.0	80.0	1.49	1.44	1.15	0.78	Parry (1973)
"	-○-	"	1.00	1.3	61.0	0.98	1.03	0.65	0.37	"
"	-○-	"	1.00	1.6	50.0	0.78	0.81	0.45	0.14	"
"	-○-	"	1.00	2.3	35.0	0.44	0.51	0.11	-.15	"
"	⏀	"	0.64	1.0	79.0	2.77	1.11	2.43	0.45	"
"	⏀	"	0.64	1.4	55.0	0.59	1.07	0.26	0.41	"
"	⏀	"	0.64	2.0	40.0	0.32	0.94	-.02	0.27	"
"	⏀	"	0.64	2.6	30.0	0.25	0.71	-.08	0.04	"
Weald Clay	○	R	1.00	1.0	30.0	0.95	1.28	0.62	0.61	Parry (1960)
"	○	"	1.00	1.0	60.0	1.01	1.28	0.68	0.61	"
"	○	"	1.00	1.0	120.	0.98	1.29	0.64	0.62	"
"	○	"	1.00	2.0	30.0	0.29	0.48	-.04	-.19	"
"	○	"	1.00	2.0	60.0	0.27	0.48	-.07	-.19	"
"	○	"	1.00	4.0	15.0	0.01	0.10	-.33	-.57	"
"	○	"	1.00	4.0	30.0	-.02	0.18	-.35	-.49	"
"	○	"	1.00	1.7	70.5	.037	0.63	0.04	-.04	"
"	○	"	1.00	8.0	15.0	-.10	-.01	-.43	-.67	"
"	○	"	1.00	12.	5.0	-.27	-.14	-.60	-.81	"
"	○	"	1.00	12.	10.0	-.30	-.05	-.63	-.71	"
"	○	"	1.00	24.	5.0	-.34	-.13	-.68	-.79	"

Table 3.1 (Continued)

SOIL	SYMBOL USED	SAMPLE PREP	K_c	OCR	σ_c' (psi)	A_f TC	A_f TE	$a_f\sqrt{2}$ TC	$a_f\sqrt{2}$ TE	REFERENCE
Weald Clay	-○-	R	1.00	1.0	120.	0.94	1.28	0.61	0.61	Badillo (1964)
"	-○-	"	1.00	1.7	70.0	0.44	0.63	0.11	-.04	"
"	-○-	"	1.00	2.0	60.0	0.34	0.47	0.00	-.19	"
"	-○-	"	1.00	4.0	30.0	-.02	0.14	-.36	-.52	"
"	-○-	"	1.00	8.0	15.0	-.22	-.06	-.55	-.73	"
"	-○-	"	1.00	12.	10.0	-.27	-.13	-.60	-.80	"
"	-○-	"	1.00	24.	5.0	-.35	-.12	-.68	-.79	"
Sault St Marie	△	R	1.00	1.0	28.4	0.48	1.07	0.15	0.40	Wu et al. (1963)
"	△	"	1.00	1.0	56.9	0.62	1.08	0.29	0.42	"
Connecticut Clay	△	UPS	0.68	1.0	42.4	1032	1024	0.99	0.58	Ladd (1972)
"	△	"	0.70	1.0	56.0	1.42	1.19	1.09	0.52	"
Boston Blue Clay	-△-	CS	0.52	1.0	-	0.60	1.17	0.27	0.50	Ladd (1971)

COMPARISON OF PORE WATER PRESSURE COEFFICIENTS FROM TRIAXIAL COMPRESSION AND EXTENSION TESTS

Values of the two forms of the pore pressure coefficient were compared to determine which, if either of them are more unique for triaxial compression and triaxial extension. Values of the pore pressure coefficient A_f calculated from triaxial compression tests are plotted in Figure 3.1 versus the values calculated from triaxial extension tests. The only difference between the values of A_f for triaxial compression and extension tests should be due to the loading path as all other test variables were the same. A forty-five degree line is drawn in Figure 3.1 indicating what would be identical values of A_f for triaxial compression and triaxial extension tests. The results of a linear regression analysis is shown as a broken line. The data presented Figure 3.1 show that the pore pressure coefficient, A_f, is not identical for the two loading paths examined (triaxial compression and triaxial extension). For the data shown, the relationship between A_f for compression and extension nearly parallels the 45 degree line as indicated by a slope of 0.99 for the linear regression line. Over 80 percent of the values of A_f shown in Figure 3.1 for triaxial extension exceed the corresponding values for triaxial compression. The average difference between values of the pore pressure coefficient, A_f, calculated for triaxial compression and triaxial extension is .19.

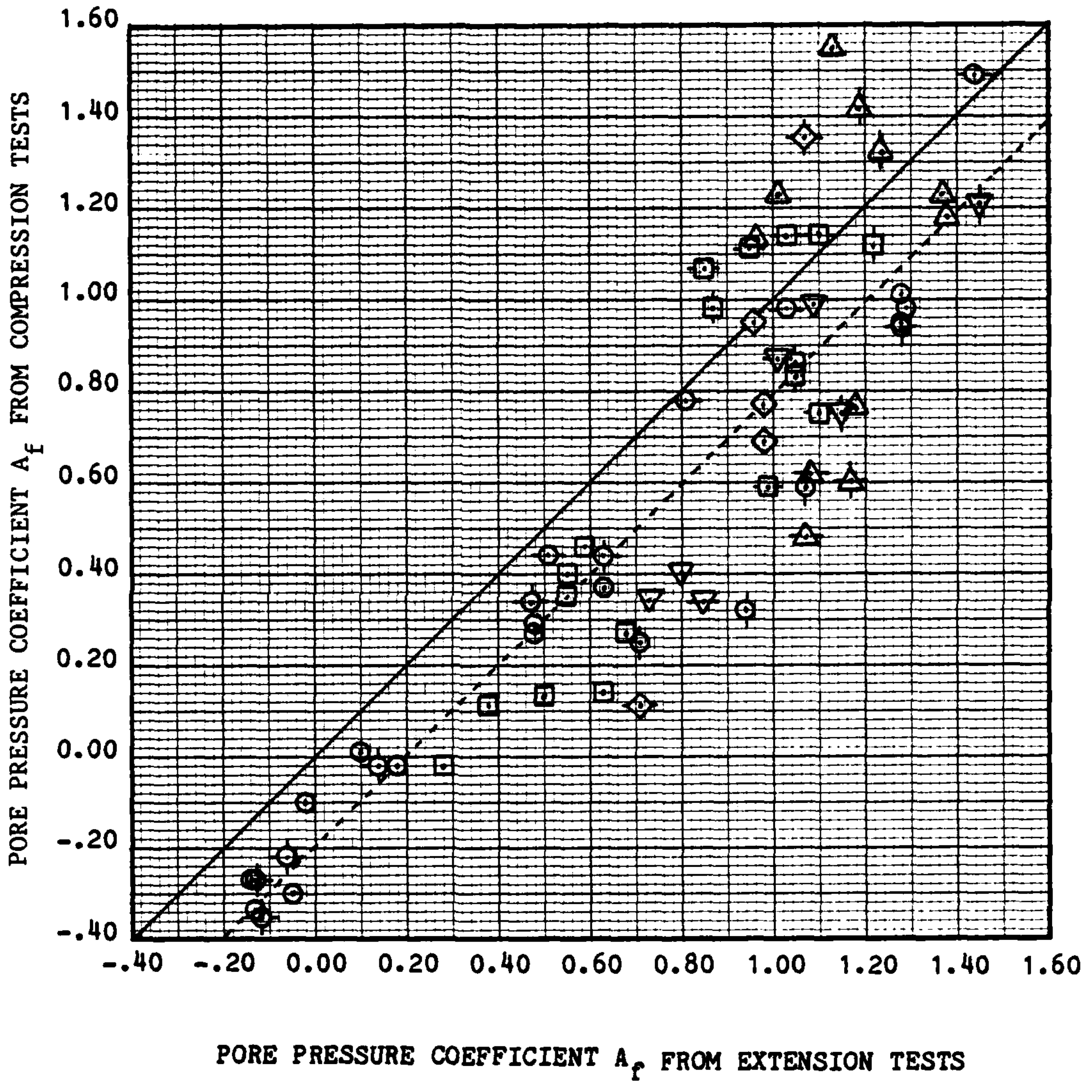


Fig 3.1 Comparison of Values of the Pore Pressure Coefficient A_f Calculated from Compression and Extension Test Data.

Calculated values of $a_f \sqrt{2}$ from triaxial compression tests are plotted versus the values from triaxial extension tests in Figure 3.2. A 45 degree, solid line is shown in this figure to indicate what would be identical values of $a_f \sqrt{2}$ for triaxial extension and compression. The broken line which is shown was obtained by a linear regression analysis of the data. The slope of the regression line is .99. Over 70 percent of the values of $a_f \sqrt{2}$ for triaxial compression exceed the corresponding value of $a_f \sqrt{2}$ for triaxial extension. The average difference between values of $a_f \sqrt{2}$ calculated for a given comparison in triaxial compression and triaxial extension is .14.

The comparisons of the pore pressure coefficients for triaxial compression and triaxial extension for both A_f and $a_f \sqrt{2}$ show considerable scatter about the regression lines. However, the comparisons of the pore pressure coefficient based on octahedral stresses, $a_f \sqrt{2}$, shows slightly closer agreement with the forty-five degree line and thus indicates that values of the pore pressure coefficient for triaxial compression and triaxial extension are more similar.

EVALUATION OF POTENTIAL FACTORS INFLUENCING THE DIFFERENCES BETWEEN VALUES OF THE PORE PRESSURE COEFFICIENT IN COMPRESSION AND EXTENSION

The data summarized in Table 3.1 represent a diverse combination of soil types, specimen preparation and sampling procedures, and stress histories prior to shear. In view of these

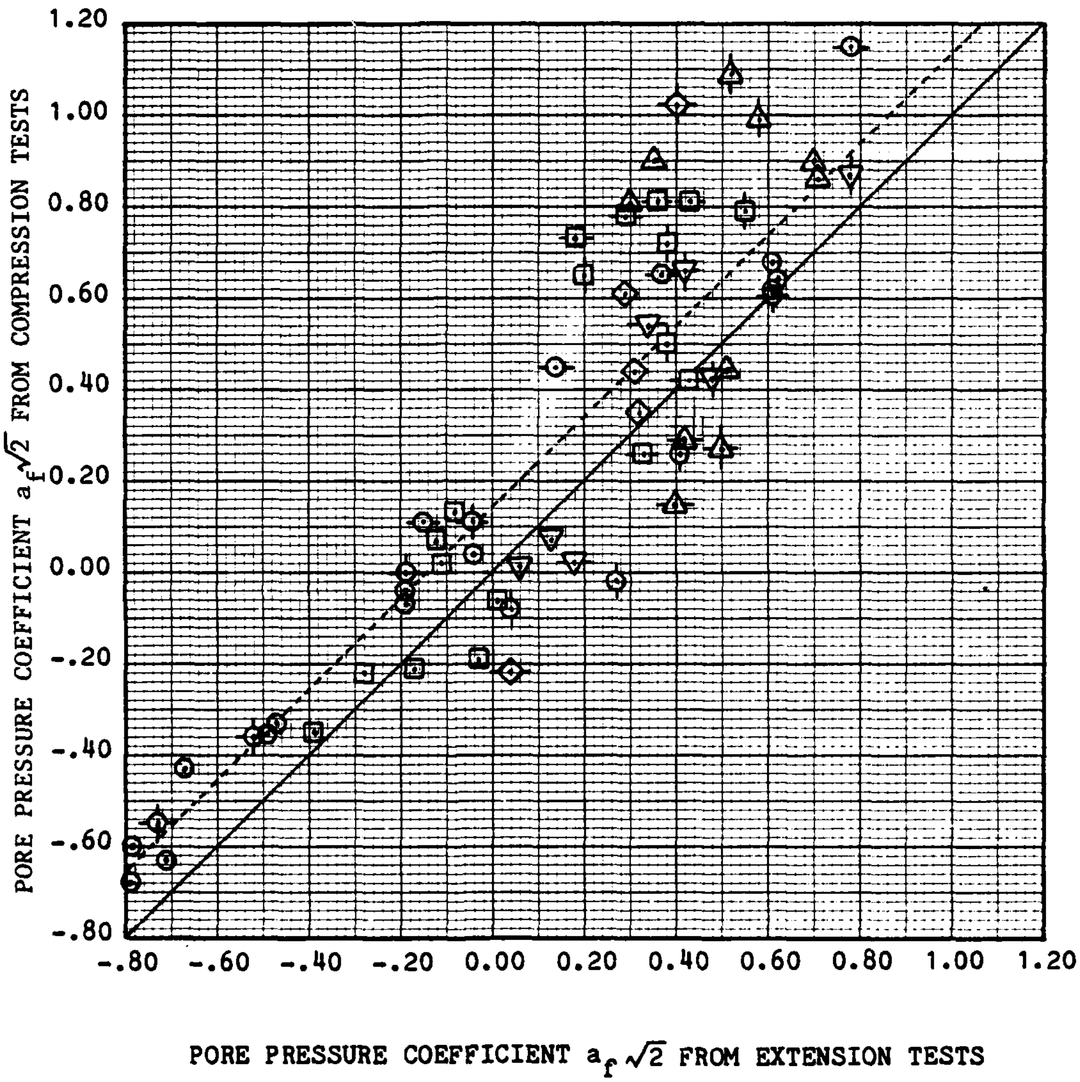


Fig 3.2 Comparison of Values of the Pore Pressure Coefficient $a_f\sqrt{2}$ Calculated from Compression and Extension Test Data.

differences, the data were examined to determine if any of these variables affected the relationship between the pore water pressure coefficients for triaxial compression and triaxial extension.

Strain at Failure.

The values of the pore pressure coefficient shown previously were all calculated at the maximum principal stress difference and the strain corresponding to this point was typically different for triaxial compression and triaxial extension tests. Therefore, the difference in the values of $a_f \sqrt{2}$ might be explained by the differences in strain at failure. To examine this possibility, the difference between the pore pressure coefficients measured in triaxial compression and in triaxial extension was plotted versus the ratio of the corresponding axial strain at failure for compression and extension as shown in Figure 3.3. The average difference in $a_f \sqrt{2}$ values is .14 as reported previously and this is shown as a solid line in Figure 3.3. It can be seen that, irrespective of the strain ratio at failure, values of the difference in $a_f \sqrt{2}$ are equally distributed above and below the average value line and no trends are apparent. Therefore, the difference in $a_f \sqrt{2}$ for triaxial compression and triaxial extension does not seem to be related to differences in the strain at failure for the two different loading paths.

Initial Effective Principal Stress Ratio, K_c.

Tests performed on both isotropically and anisotropically

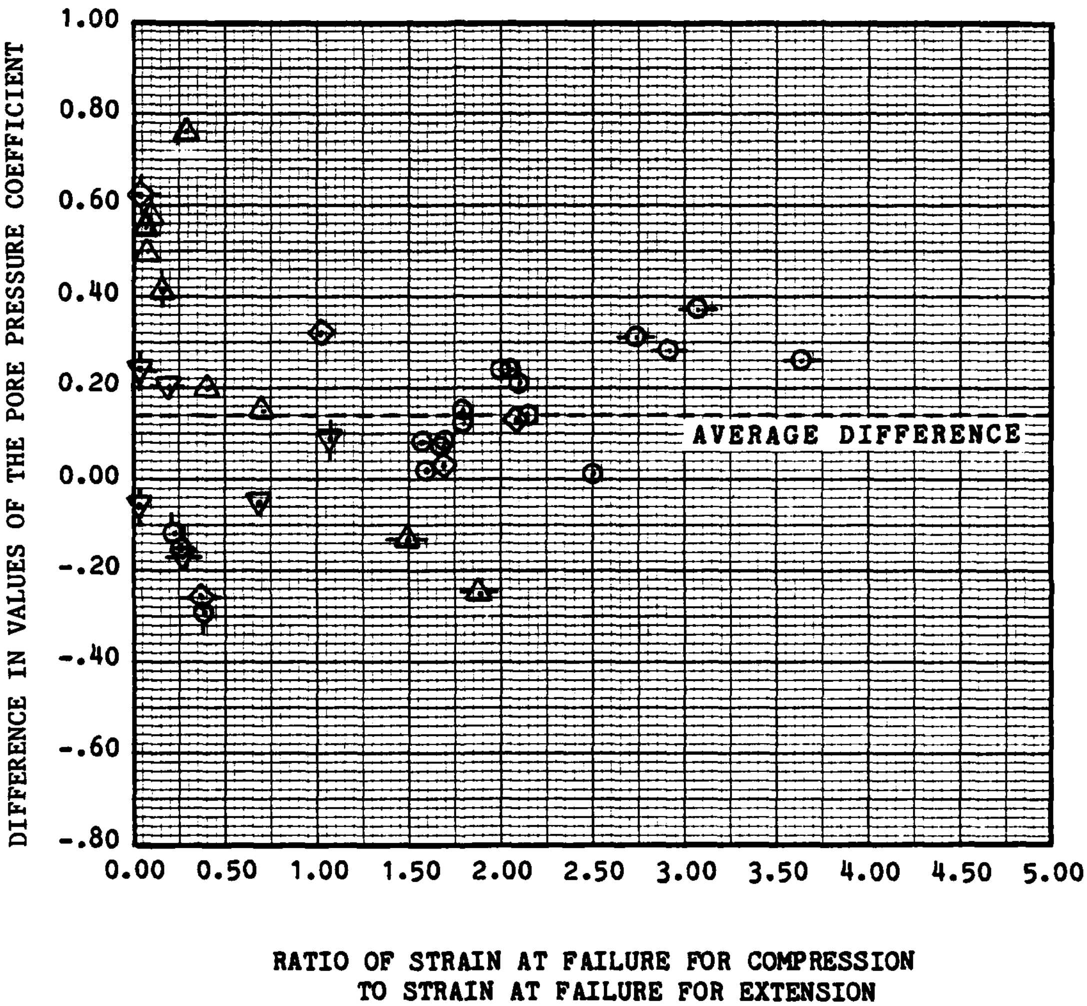


Fig 3.3 Comparison of the Difference in Values of the Pore Pressure Coefficient $a_f\sqrt{2}$ with the Ratio of Strain at Failure Calculated from Compression and Extension Test Data.

consolidated specimens are included in the data summarized in Table 3.1. To determine if the type of consolidation (isotropic or anisotropic) may have had an effect on the relative values of $a_f\sqrt{2}$ the data were plotted in Figure 3.4 and points representing anisotropically consolidated samples were highlighted by using solid symbols. A nearly equal distribution of values above and below the 45 degree line is seen for specimens consolidated both isotropically and anisotropically. Thus, the data show no difference in $a\sqrt{2}$ values which can be directly related to the effective principal stress ratio used for consolidation.

Overconsolidation Ratio.

The data presented in Table 3.1 represented a range in overconsolidation ratio (OCR) which is defined as:

$$OCR = \frac{\sigma_{vm}'}{\sigma_{vo}'} \qquad 3.1$$

where σ_{vm}' is the maximum vertical effective stress the soil has been subjected to and σ_{vo}' is the current vertical effective stress. Experimental results by Henkel (1956), Parry (1960), Bjerrum and Simons (1960), and Perloff and Osterberg (1964) indicate that, for cohesive soils, the value of the pore pressure coefficient is a function of the overconsolidation ratio. It is possible that the difference in values of the pore pressure coefficient for different loading paths may also be a function of the overconsolidation ratio. To examine this possibility, the difference in $a_f\sqrt{2}$ for triaxial

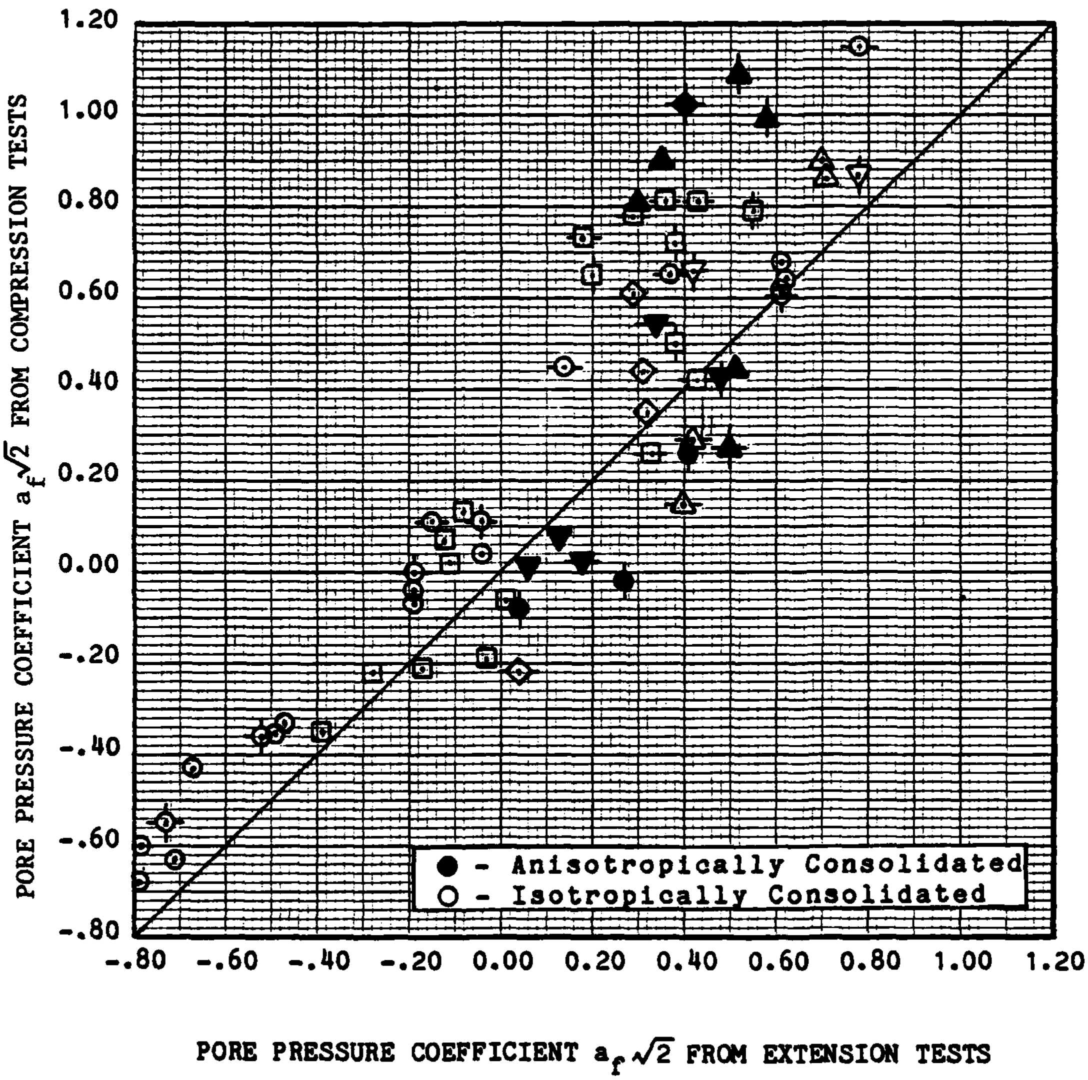


Fig 3.4 Influence of Consolidation on the Relationship of the Values of the Pore Pressure Coefficient $a_f\sqrt{2}$ Calculated from Compression and Extension Test Data.

compression and extension was plotted versus the overconsolidation ratio as shown in Figure 3.5. Except for the weathered Bangkok clay, there is remarkable consistency in the difference in $a_f\sqrt{2}$ values across the entire range in overconsolidation ratios at which these soils were tested. The inconsistencies noted in the data for the weathered Bangkok Clay are most probably due to changes in the soil structure which were caused by consolidating the specimens to pressures greater than the maximum previous pressure. The majority of the data show that the differences in $a_f\sqrt{2}$ values are not affected by the degree of overconsolidation.

Confining Pressure.

Separate from the effects of overconsolidation on the differences in $a_f\sqrt{2}$ are the possible effects of confining stress at a given overconsolidation ratio. To explore this possibility, data were selected from Table 3.1 for cases where tests were conducted on the same soil at the same overconsolidation ratio but at different confining pressures. These data are presented in Figure 3.6 which shows the difference between values of $a_f\sqrt{2}$ for triaxial compression and triaxial extension versus confining pressure for five different soil types. The data show no consistent trends which would indicate that the differences in $a_f\sqrt{2}$ values are affected by confining pressure.

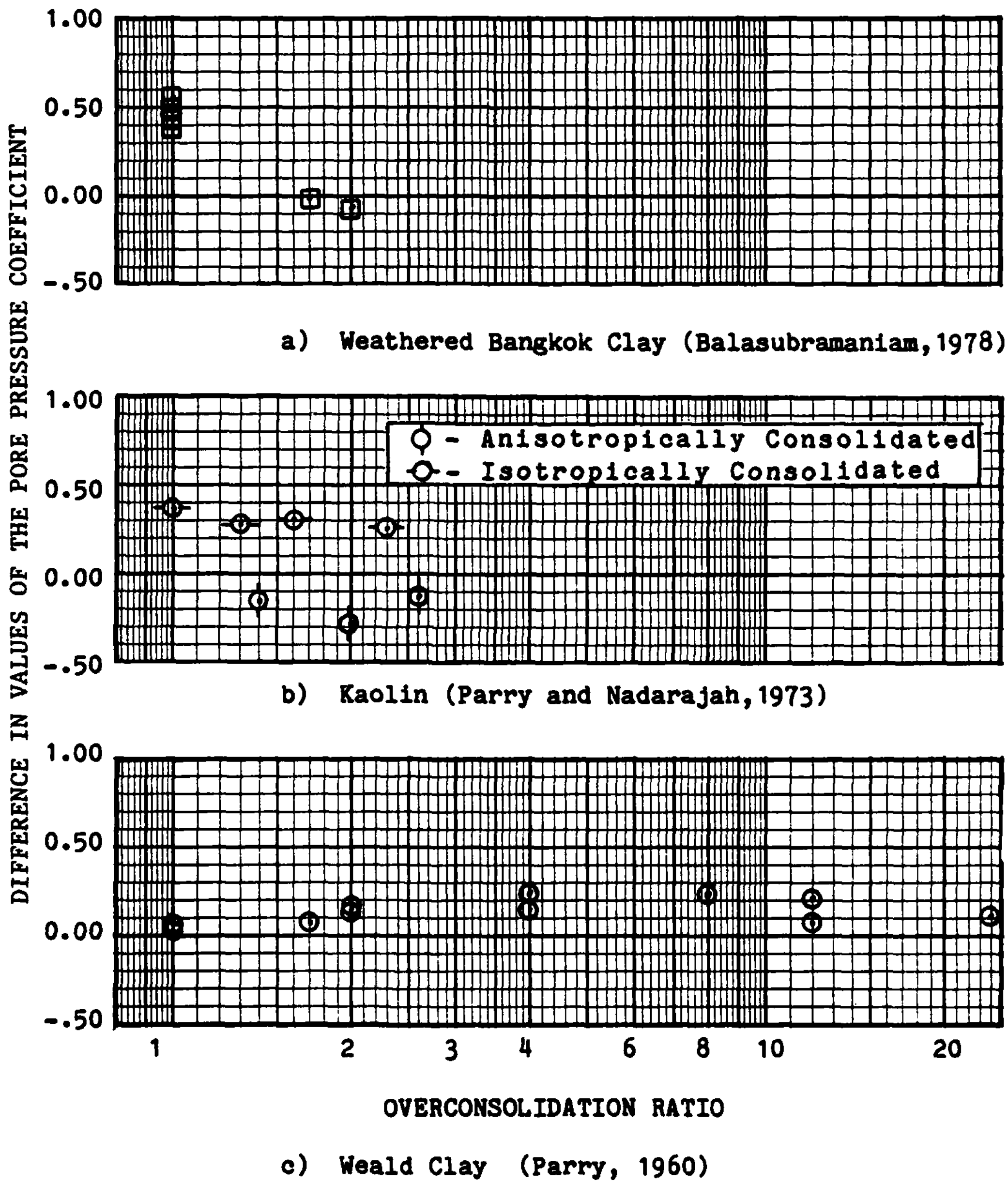


Fig 3.5 Effect of Overconsolidation on the Difference in the Values of the Pore Pressure Coefficient $a_f\sqrt{2}$ Calculated from Compression and Extension Test Data.

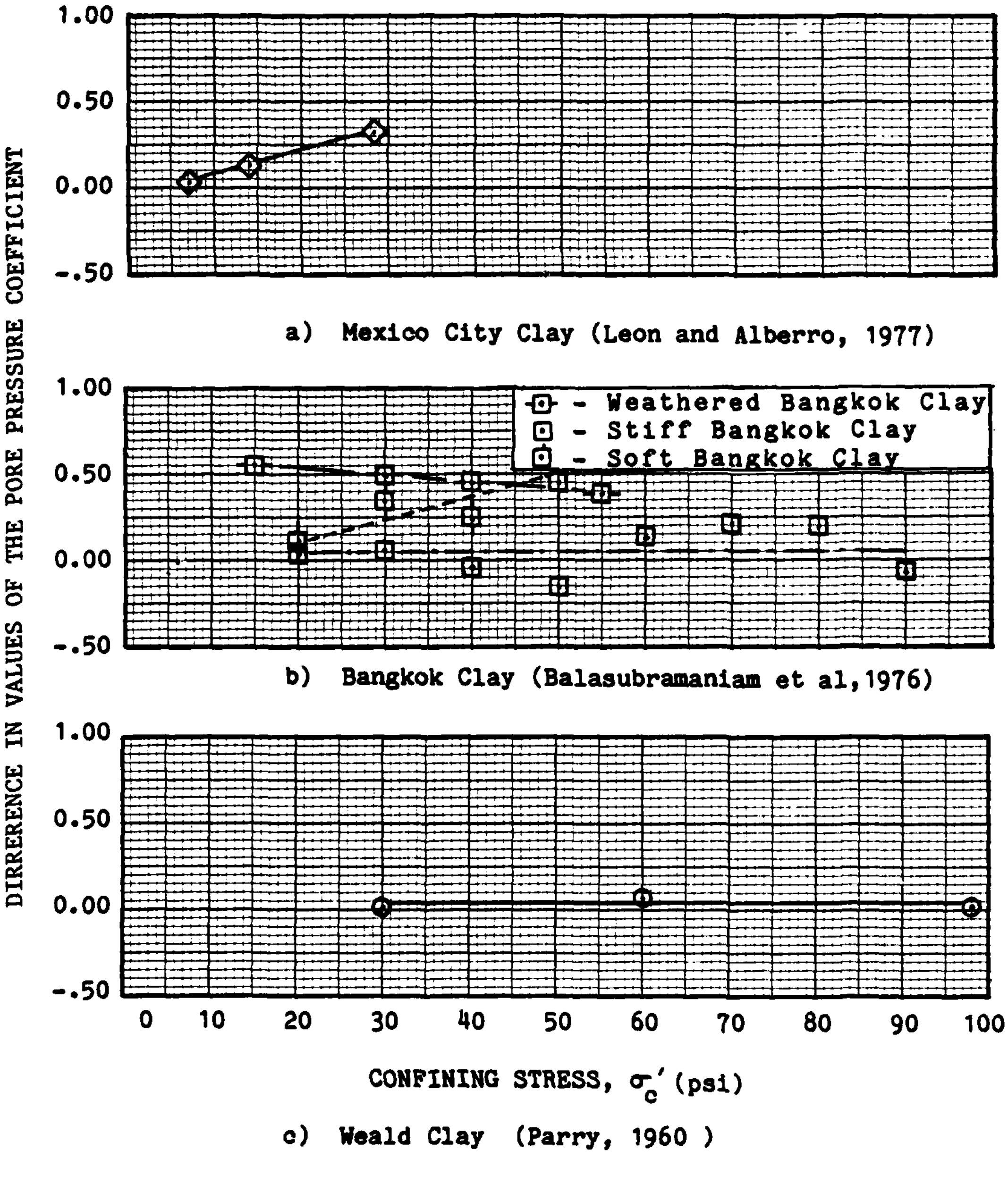


Fig 3.6 Effect of Confining Pressure on the Difference in the Values of the Pore Pressure Coefficient $a_f\sqrt{2}$ Calculated from Compression and Extension Test Data.

<u>Sample Preparation Procedure.</u>

The data presented in Table 3.1 represented a variety of specimen preparation procedures. To determine if specimen preparation procedure may have had an affect on the relation of the values of $a_f\sqrt{2}$ in triaxial compression and extension, the data have been plotted in Figure 3.7 with the data points for tests conducted on remolded specimens shown as solid symbols. This has been repeated in Figure 3.8 with the data points for tests conducted on specimens prepared from consolidated slurries of the soil and again in Figure 3.9 for specimens prepared from undisturbed samples of soil. The data presented in Figures 3.7, 3.8, and 3.9 show no trends to indicate that specimen preparation procedure had an effect on the relative values of the pore pressure coefficient in triaxial compression and extension.

INFLUENCE OF PLANE STRAIN ON THE VALUE OF THE PORE PRESSURE COEFFICIENT

Plane strain represents a loading which is different from either triaxial compression or triaxial extension. In addition, plane strain often more closely represents the deformation conditions at failure for earth slopes. Since a major objective of this research was to evaluate the pore pressure coefficients for use in analyses of earth slopes, the influence of plane strain on the value of the pore pressure coefficient was of considerable interest. Three distinct approaches were taken to evaluate the influence of plane strain on the

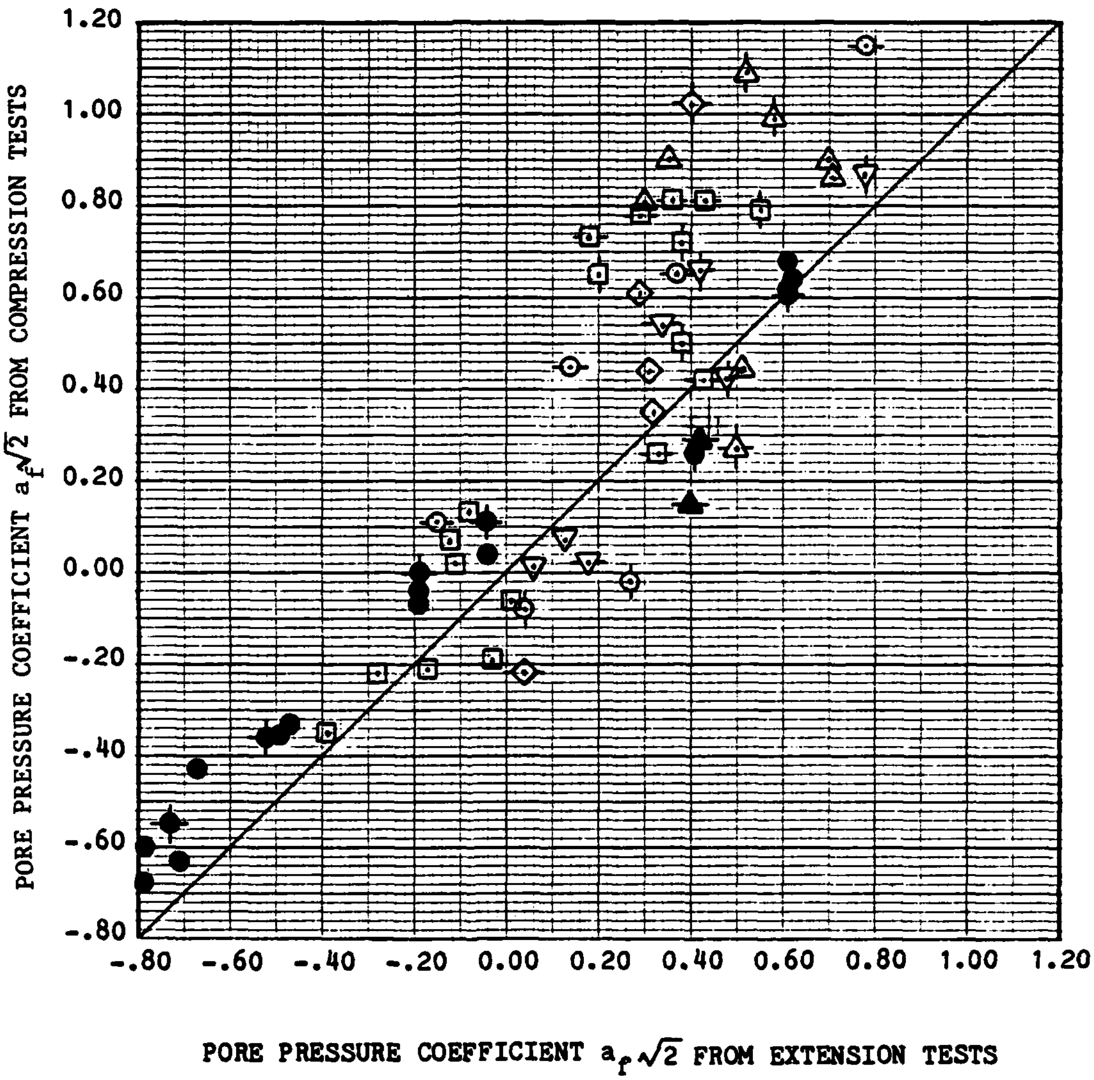


Fig 3.7 Comparison of Values of the Pore Pressure Coefficient $a_f\sqrt{2}$ Calculated from Compression and Extension Test Results with Data for Specimens Prepared From Remolded Samples Highlighted.

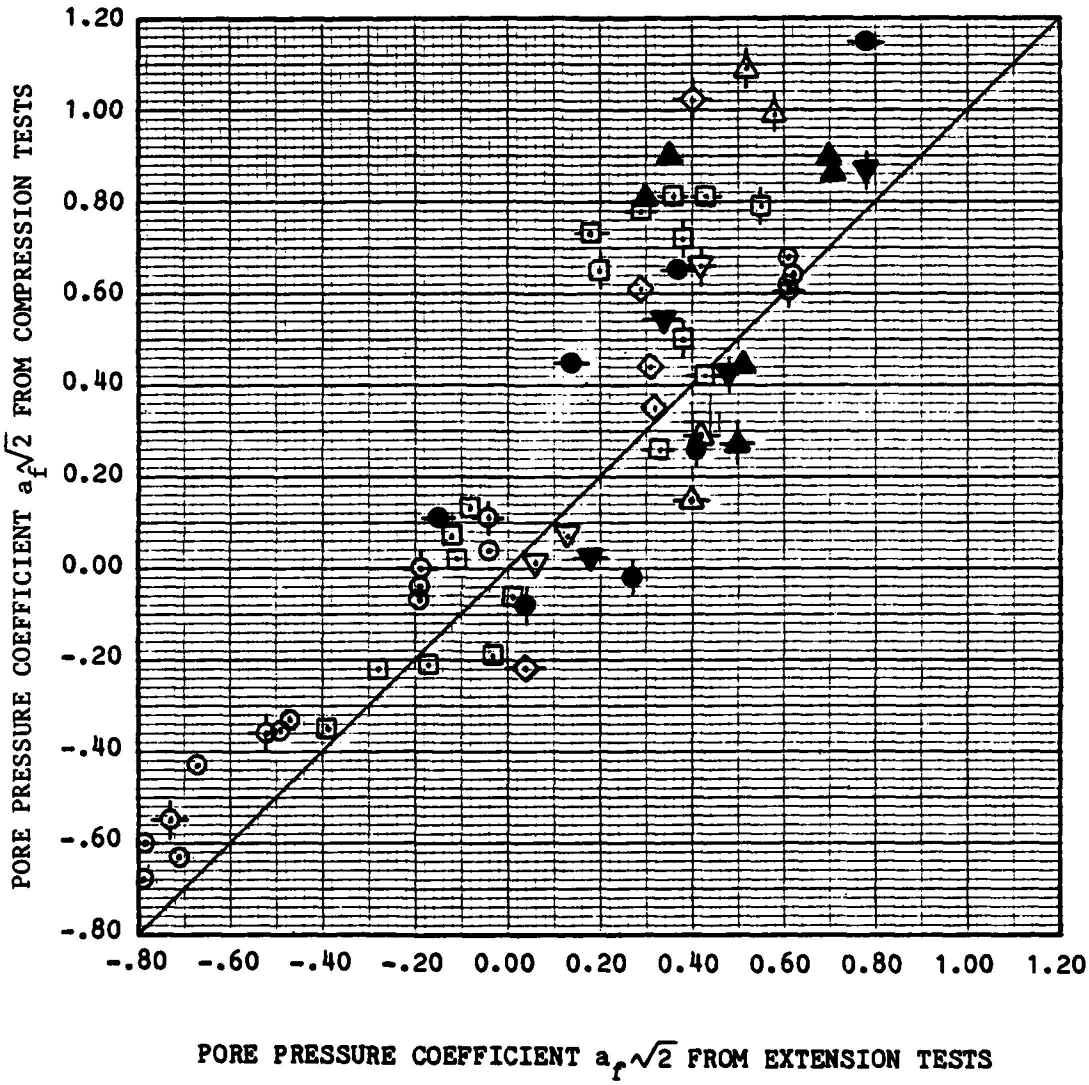


Fig 3.8 Comparison of Values of the Pore Pressure Coefficient $a_f\sqrt{2}$ Calculated from Compression and Extension Test Results with Data for Specimens Prepared from a Consolidated Slurry Highlighted.

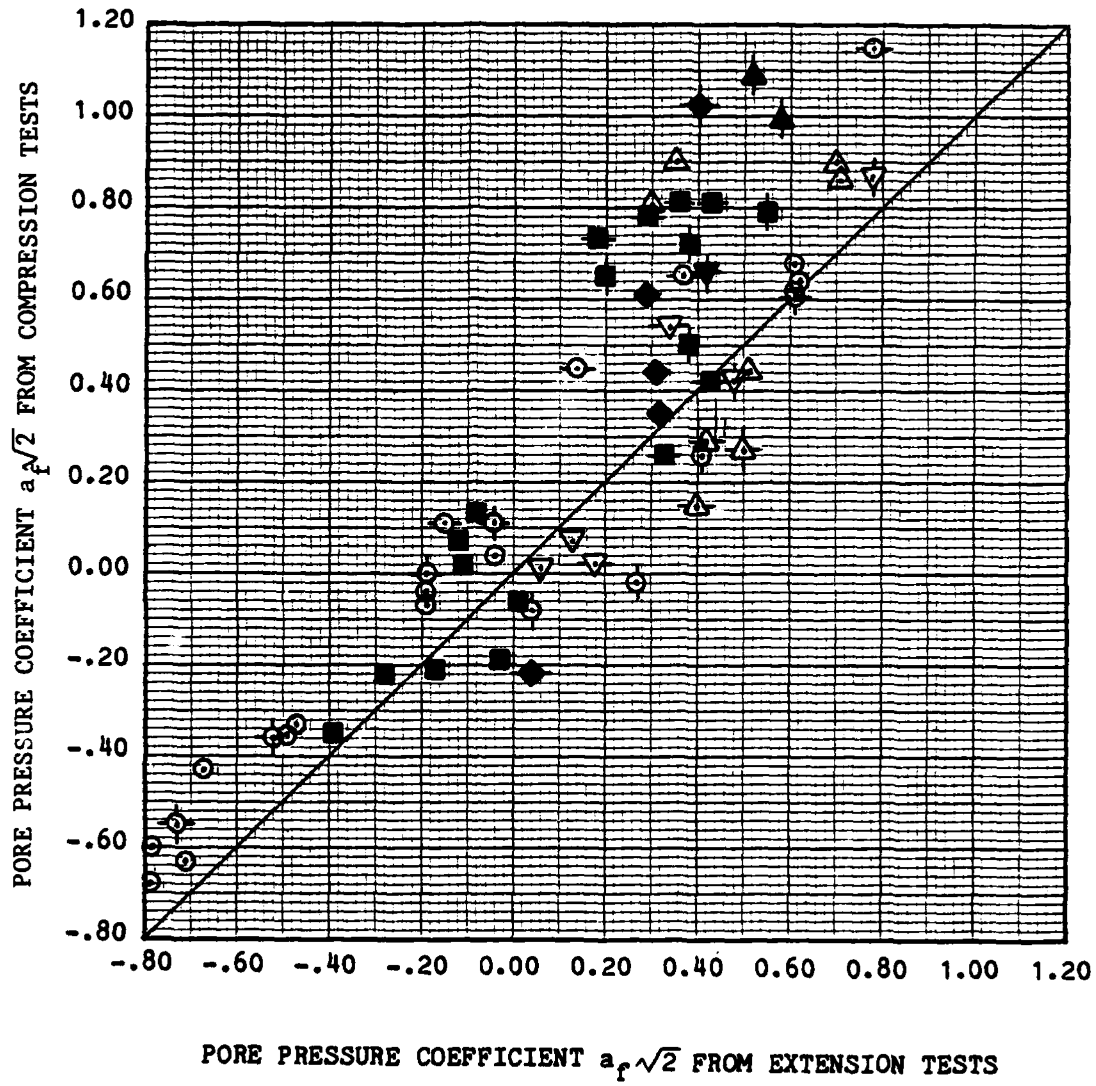


Fig 3.9 Comparison of Values of the Pore Pressure Coefficient $a_f\sqrt{2}$ Calculated from Compression and Extension Test Results with Data for Specimens Prepared From Undisturbed Samples Highlighted.

pore pressure coefficients. First, values of the pore pressure coefficient calculated from the results of plane strain compression and plane strain extension tests were compared. Second, values of the pore pressure coefficient calculated from plane strain tests were compared with values of the pore pressure coefficient calculated from the results of triaxial tests. Third, the effect of the magnitude of the intermediate principal change in stress on values of the pore pressure coefficient was examined.

Comparison of A_f for Different Plane Strain Loading Paths

A survey of the literature revealed that there are relatively few instances in which values of the intermediate principal stress are reported for plane strain tests. However it is of interest to examine how values of the pore pressure coefficient, A_f, calculated from plane strain tests vary with loading path even though the magnitude of the intermediate principal change in stress is not known. Values of A_f calculated from plane strain compression tests are plotted versus values calculated from plane strain extension tests in Figure 3.10. The differences between the values of A_f for the compression and extension tests should be due solely to the loading path used since all other test variables were the same for each pair of tests compared. A forty-five degree line is drawn in Figure 3.10 to indicate what would be identical values of A_f for plane strain compression and plane strain extension. The data presented in Figure 3.10 show considerable scatter about the forty-five degree line and

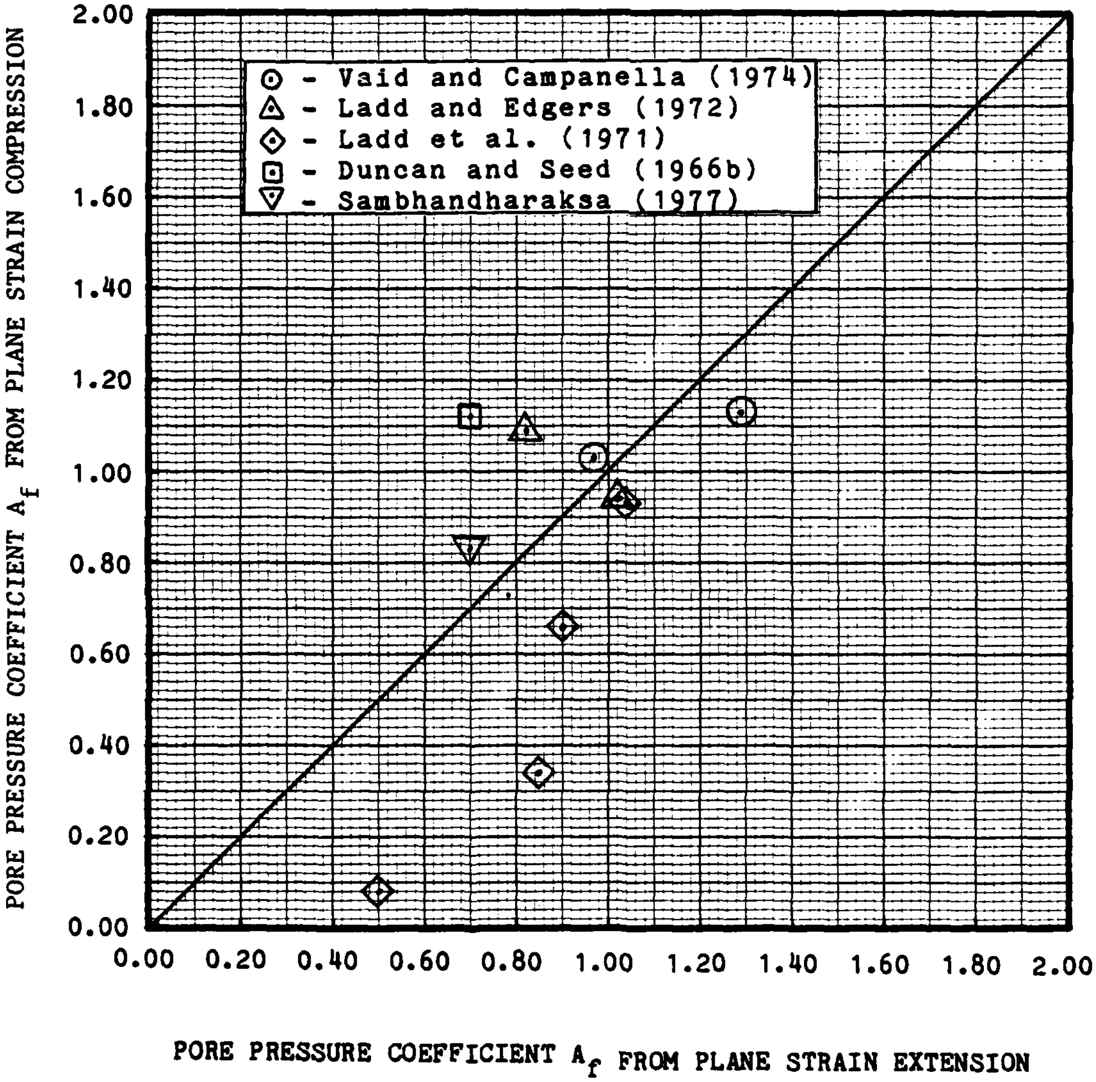


Fig 3.10 Comparison of Values of the Pore Pressure Coefficient A_f Calculated from Plane Strain Compression and Extension Test Results.

indicate that values of the pore pressure coefficient are not identical for the two plane strain loading paths examined. This finding is consistent with findings reported earlier in which A_f values were compared for triaxial compression and extension tests.

Comparison of A_f Values for Plane Strain with Values for Triaxial Tests

Triaxial compression and triaxial extension represent extremes for values of the intermediate principal change in stress, $\Delta\sigma_2$ ($\Delta\sigma_2 = \Delta\sigma_1$ or $\Delta\sigma_2 = \Delta\sigma_3$). However, Cornforth (1964) showed that the angle of internal friction measured in plane strain for sands is not bounded by triaxial compression and extension. To determine if Cornforth's findings might also apply to the pore pressure coefficient, values of the pore pressure coefficient, A_f calculated from plane strain test data are plotted versus values calculated from triaxial test data in Figure 3.11. A forty-five degree line is drawn in Figure 3.11 indicating what would be identical values of A_f for plane strain and triaxial shear. Solid symbols indicate comparisons of the pore pressure coefficient for extension tests while open symbols reflect comparisons of A_f for compression tests. The data presented in Figure 3.11 show considerable scatter about the forty-five degree line. However, the data seems to indicate that triaxial compression and triaxial extension may not define the upper and lower bounds on possible values of the pore pressure coefficient.

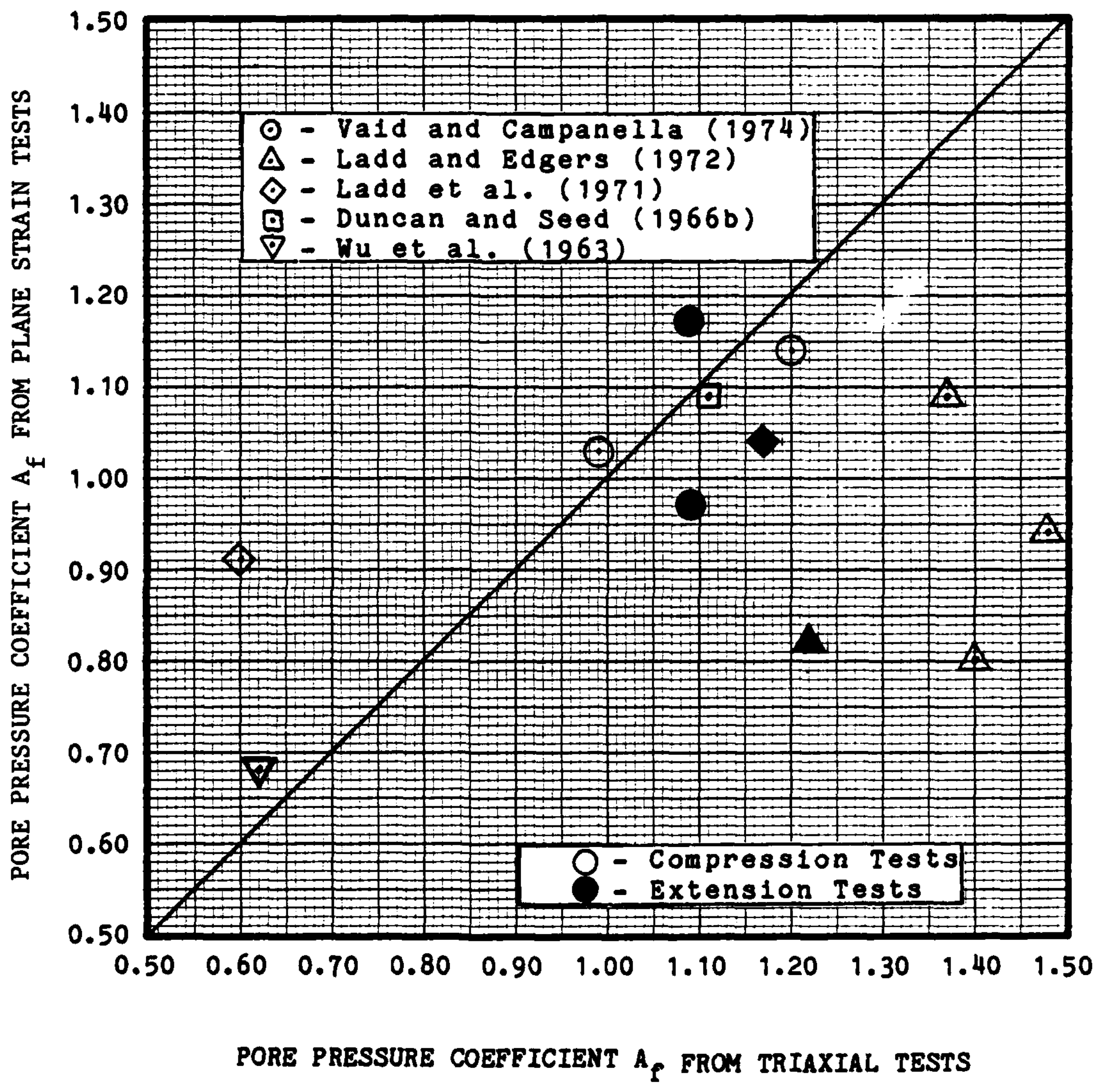


Fig 3.11 Comparison of Values of the Pore Pressure Coefficient A_f Calculated from Plane Strain and Triaxial Test Results.

Influence of the Intermediate Principal Change in Stress on $a_f\sqrt{2}$

Expression of the pore pressure coefficient in terms of octahedral stresses implies that the pore pressure changes induced in a soil specimen during undrained shear are dependent upon the intermediate principal change in stress. Therefore, it is of interest to examine the effect of the intermediate principal change in stress on the value of the pore pressure coefficient, a_f. Ladd et al. (1971) conducted a series of plane strain tests on specimens of Boston Blue Clay prepared from consolidated slurries. The specimens were anisotropically consolidated (K_o = 0.50) and the loading paths used included axial compression, axial extension, and lateral compression. Values of the pore pressure coefficient, $a_f\sqrt{2}$ are plotted in Figure 3.12 versus the Intermediate Principal Stress Index, b. The Intermediate Principal Stress Index, as proposed by Bishop (1971), is expressed as:

$$b = \frac{\Delta\sigma_2 - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3} \qquad 3.2$$

A value of b equal to zero indicates axially symmetric compression loading ($\Delta\sigma_2 = \Delta\sigma_3$) while a value of b equal to 1 indicates axially symmetric extension loading ($\Delta\sigma_2 = \Delta\sigma_1$). The intermediate principal change in stress has been expressed in terms of the Index b because it provides a convenient means to relate the intermediate principal change in stress to the major and minor principal changes in stress. The data presented in Figure 3.13 indicates that there is no

significant trend in the way the pore pressure coefficient varies with the Intermediate Principal Stress Index.

Wu et al. (1963) conducted a series of triaxial tests on hollow cylinder specimens of remolded Sault Saint Marie Clay. Values of $a_f\sqrt{2}$ and b were calculated based on tabulated test results presented by the authors. Values of $a_f\sqrt{2}$ for the Sault Saint Marie Clay are plotted versus b in Figure 3.13. Again, there is no significant trend in the variation of $a_f\sqrt{2}$ with b. Although the preceding evaluation represents very few plane strain tests for which values of the intermediate principal stress were reported, the available data seem to indicate that there is not a consistent trend in the variation of $a_f\sqrt{2}$ with the intermediate principal change in stress.

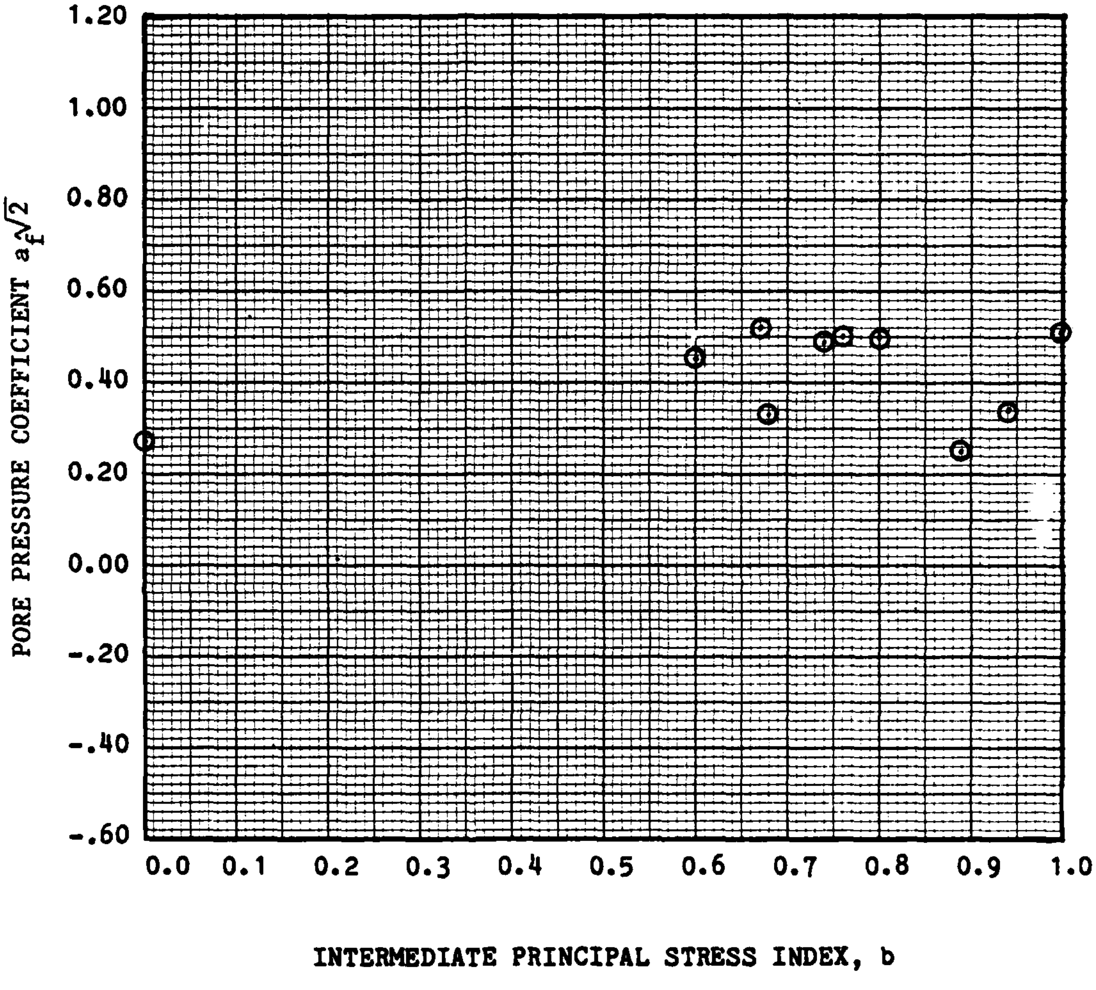


Fig 3.12 Variation in the Pore Pressure Coefficient, $a_f\sqrt{2}$ with the Intermediate Principal Stress Index, b for Plane Strain Tests Conducted on Boston Blue Clay

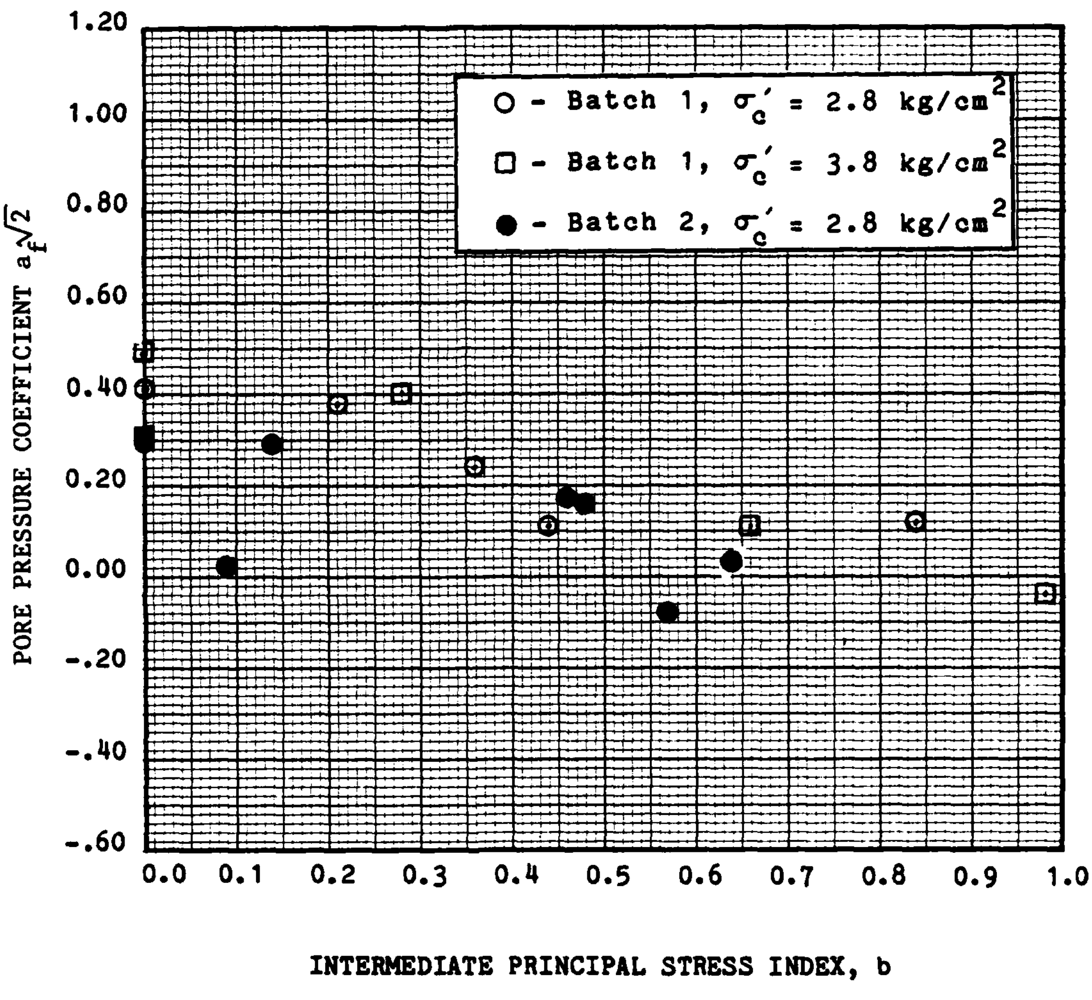


Fig 3.13 Variation in the Pore Pressure Coefficient $a_f\sqrt{2}$ with the Intermediate Principal Stress Index, b for Hollow Cylinder Triaxial Tests on Sault Saint Marie Clay

CONCLUSIONS

The two forms of the pore pressure coefficient, Skempton's pore pressure coefficient, A_f, and the pore pressure coefficient based on octahedral stresses, $a_f \sqrt{2}$, were evaluated to determine which, if either, is relatively independent of loading path. In comparisons conducted using a data set representing 140 triaxial tests on 18 different soils, it was shown that neither form of the pore pressure coefficient is uniquely independent of loading path. The differences found in both forms of the pore pressure coefficient for triaxial compression and triaxial extension did not appear to be related to strain at failure, initial effective principal stress ratio, overconsolidation ratio, confining pressure, or type of specimen. An evaluation of the influences of plane strain values of the pore pressure coefficient, A_f indicated that triaxial compression and triaxial extension do not establish upper and lower bounds on possible values of the pore pressure coefficient. In addition, available data indicate that variations in values of the pore pressure coefficient do not appear to be related to the magnitude of the intermediate principal change in stress. The data did show that the difference in values of the pore pressure coefficient based on octahedral stresses are slightly smaller than the difference in values of Skempton's pore pressure coefficient for triaxial compression and triaxial extension.

CHAPTER 4

FINITE ELEMENT COMPUTATION OF STRESSES INDUCED BY RAPID DRAWDOWN

INTRODUCTION

The purpose of this study is to evaluate the use of pore pressure coefficients in the analysis of rapid drawdown in earth slopes. To use the pore pressure equation to estimate the changes in pore pressure which are induced in an earth slope as the result of rapid drawdown, the principal changes in stress must be known. Because measured data was not available in the literature, the finite element method was used to obtain an estimate of the principal changes in stress which are induced in an earth slope as the result of a rapid lowering of the reservoir level adjacent to the slope. The computer code used in this investigation is described. The assumptions made in defining the boundary and load conditions, material properties, and slope geometries are summmarized. The estimated values of the principal changes in stress obtained by this finite element analysis are presented. And, the influence which modulus and slope inclination have on the values of the major and minor principal changes in stress otained from the finite element computations is evaluated.

DESCRIPTION OF COMPUTER CODE

The Texas Grain Analysis Program (TEXGAP)(Becker and Dunham, 1979) was used to perform the finite element computations for this study. TEXGAP is a program which was developed for the analysis of two-dimensional, linearly elastic plane or axisymmetric bodies. It has been periodically updated and used extensively since its initial development in 1974 and, therefore, it provides a reliable program that has direct application to the class of problem which was to be analyzed for this study. To insure that input data had been correctly formatted, several verification runs were accomplished in which an identical problem was analyzed using TEXGAP and PRONEW, a two dimensional finite element program developed by Dr. Stephen G. Wright at the University of Texas at Austin. Following code verification, an analysis was conducted to develop a mesh geometry which would minimize computer execution time without reducing the accuracy of the analysis. The details of code verification and the mesh size analysis are presented in Appendix A. The mesh geometries used in the analysis of rapid drawdown are shown in Figure 4.1a for a 2:1 upstream slope and in Figure 4.1b for a 4:1 slope. A graduated mesh was selected with the finer portion concentrated in the upstream slope of the dam. This mesh configuration was chosen because it concentrated data points in the upstream portion of the slope which is the area of prime concern in cases of rapid drawdown.

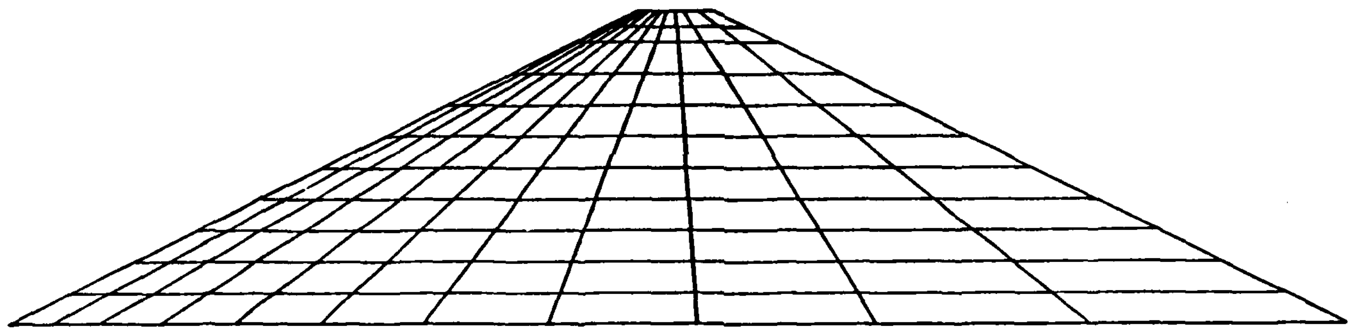


Fig. 4.1a Finite Element Mesh for a 2:1 Upstream Slope

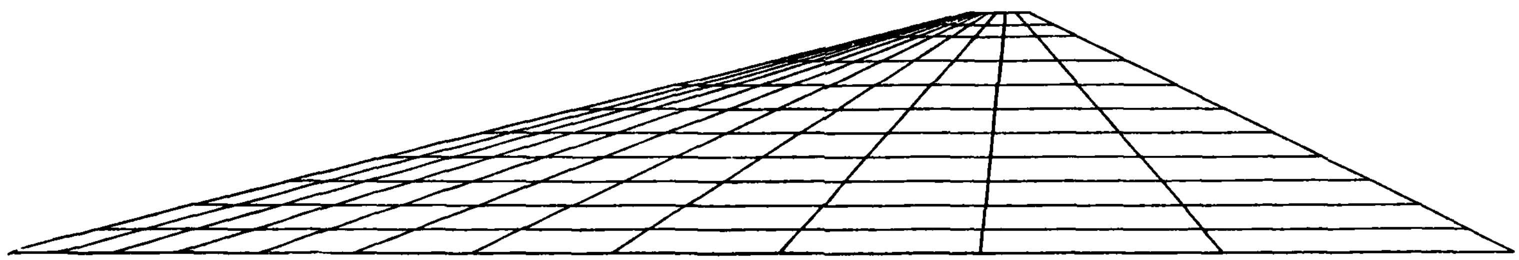


Fig. 4.1b Finite Element Mesh for a 4:1 Upstream Slope.

FINITE ELEMENT MODELING OF RAPID DRAWDOWN

Boundary and Load Conditions.

The boundary conditions employed for the finite element computations are illustrated in Figure 4.2. The earth dam was assumed to rest on a perfectly rigid, rough base. Therefore, horizontal and vertical displacements were assumed to be zero at nodal points along the base of the dam. The drop in reservoir level was assumed to occur instantaneously with an initial position at the crest of the dam and a final position at the base of the dam. The changes in stress produced by this drop were modeled as reductions in stress normal to the upstream slope of the dam. Stress free conditions were specified at nodal points along the crest and downstream face of the dam. Because the drawdown was assumed to be instantaneous, no drainage would occur within the dam and, thus, there would be no change in the unit weight of the soil. Accordingly, body forces were eliminated from the analysis.

Material Properties.

To obtain a first approximation of the principal changes in stress, linearly elastic analyses were performed. Values for Poisson's ratio, μ, and soil modulus, E_s, were assigned to each element. The loading resulting from drawdown was assumed to occur under conditions of no volume change and a Poisson's ratio value of .495 was assigned to all elements. Two sets of values of Young's

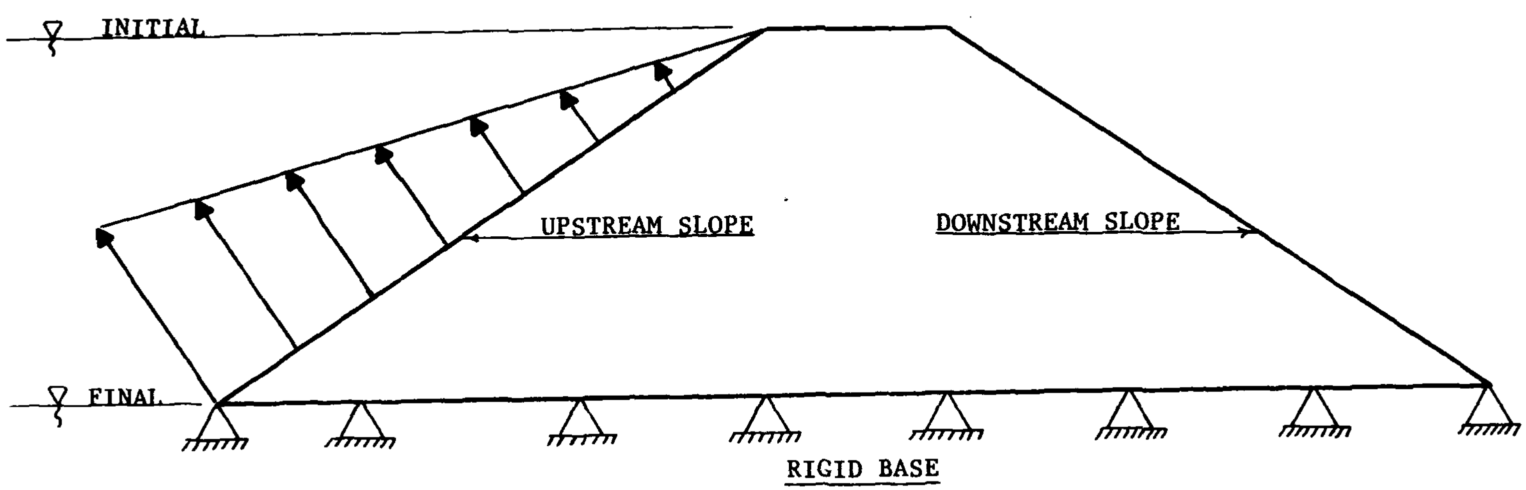


Fig. 4.2 Boundary Conditions in Finite Element Analyses of Rapid Drawdown

modulus were selected to represent probable "upper" and "lower" bounds on the variation in modulus within the embankment. In one case, a constant modulus value was assigned to each element in the mesh (E_s equal to 1,000,000, psf). In the other case, the modulus was assumed to increase linearly with depth and modulus values were assigned to each element based on the vertical depth to the geometric center of the element (E_s varied from 0 to 2,000,000 psf). A sensitivity analysis showed that the magnitude and the range in magnitude of values assumed for the modulus had no effect on the values of the principal changes in stress calculated by the finite element method. The details of that sensitivity analysis are presented in Appendix A.

Slope Geometry.

A slope height of 100 feet was used in all of the finite element computations conducted for this study. A 2:1 slope was selected for all downstream slopes in this analysis. Upstream slopes of 2:1 and 4:1 were used because they represent an upper and lower bound for all cases in which rapid drawdown induced slope failures had occurred (based on Table 1.1).

RESULTS OF FINITE ELEMENT ANALYSIS

Vectors of Major and Minor Principal Change in Stress.

Values of the principal changes in stress and their orientation with respect to horizontal were used to develop stress vectors of the major and minor principal changes in stress. The magnitude of the change in stress is represented by the scaled length of the vector and the direction is shown by drawing the vector at the specified angle from horizontal. A stress vector plot of major and minor principal changes in stress induced by rapid drawdown loading in an earth dam with a 2:1 upstream slope is shown in Figure 4.3a. A similar plot for a 4:1 upstream slope is shown in Figure 4.3b. A rapid dissipation of changes in stress is apparent in these figures such that for all practical purposes the loading only affects the upstream slope of the dam. Accordingly, only the stress changes in the upstream slope are considered further.

Influence of the Assumed Value of Soil Modulus on the Major and Minor Principal Change in Stress.

To facilitate an evaluation of the influence of modulus on the principal changes in stress throughout the slope, representative horizontal planes in the lower half (y/h = .20), at midheight (y/h = .50), and in the upper half (y/h = .80) of the slope were selected for the presentation of data where y is the vertical height of the plane above the base of the slope and h is the total height of the slope. In the

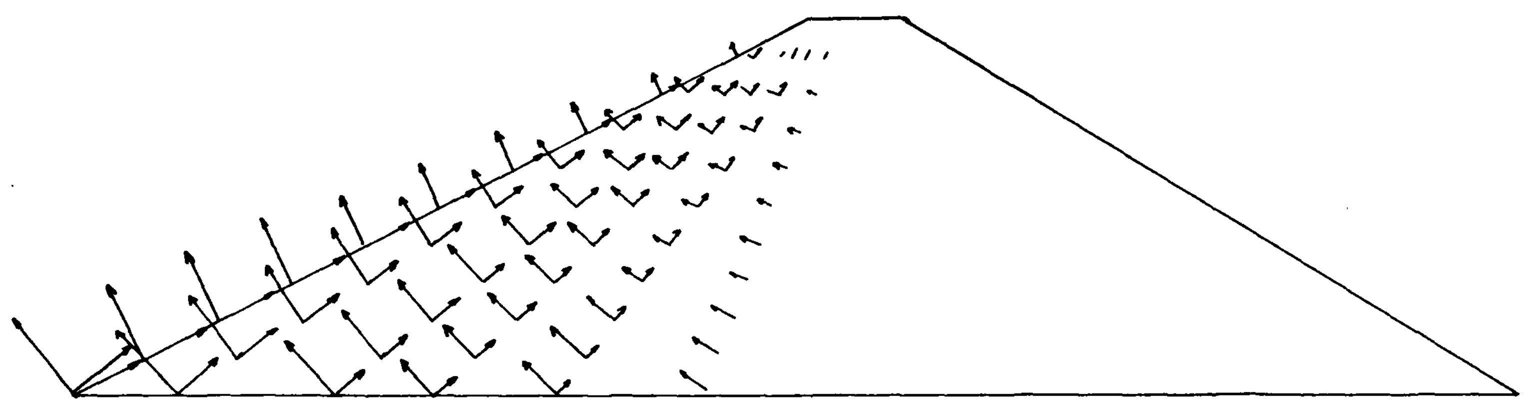


Fig. 4.3a Vectors of Major and Minor Principal Change in Stress for a 2:1 Upstream Slope

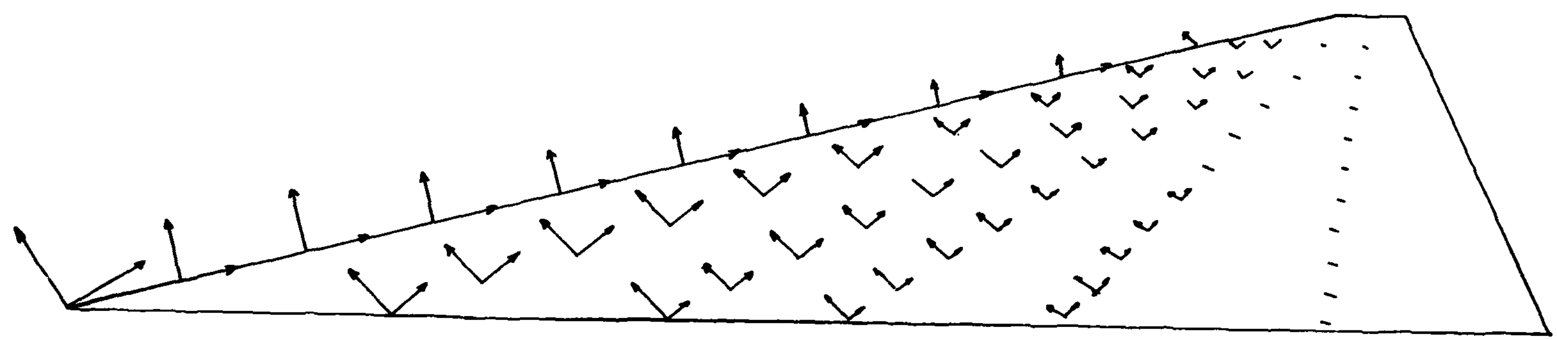


Fig. 4.3b Vectors of Major and Minor Principal Change in Stress for a 4:1 Upstream Slope

graphical presentations which follow, the changes in stress have been normalized with respect to the hydrostatic pressure, u_s, which would exist on that plane prior to drawdown assuming a horizontal water surface and no flow. The position of data points within the slope is presented in terms of a normalized horizontal position, D_h, such that:

$$D_h = x/X \tag{4.1}$$

where X is the horizontal distance from the upstream face of the slope to the crest of the dam and x is the horizontal distance from the upstream face of the slope to the data point. The variation in the normalized values of the major and minor principal changes in stress along horizontal planes through the upstream portion of a 2:1 slope is shown in Figures 4.4a through c and for a 4:1 slope in Figures 4.5a through c assuming both a constant and linearly varying modulus. The broken line shown in these figures represents the rapid drawdown induced reduction in hydrostatic pressure along the face of the slope immediately above the points on the horizontal plane being considered. These figures show that both the major and minor principal changes in stress attenuate away from the face of the slope. In addition, it can be seen that the absolute value of the minor principal change in stress is greater than and the absolute value of the major principal change in stress is less than the reduction in hydrostatic pressure along the upstream face of the slope immediately above the point considered. Both of these trends are independent of upstream slope inclination, position within the slope, and assumed variation in modulus. However, the computed values of the major principal change

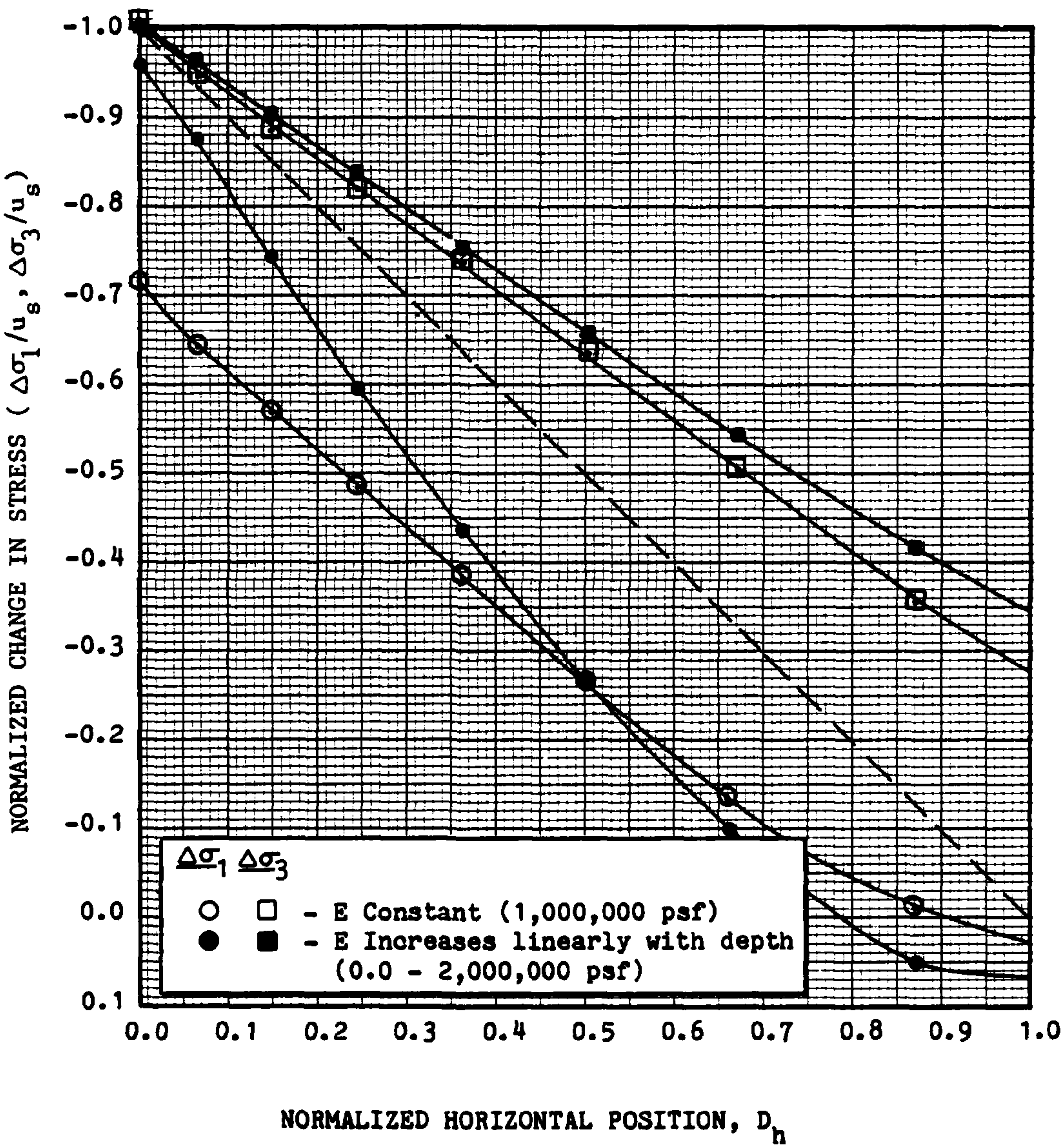


Fig. 4.4a Variation in the Major and Minor Principal Change in Stress Across a Horizontal Plane at y/h = 0.20 in a 2:1 slope.

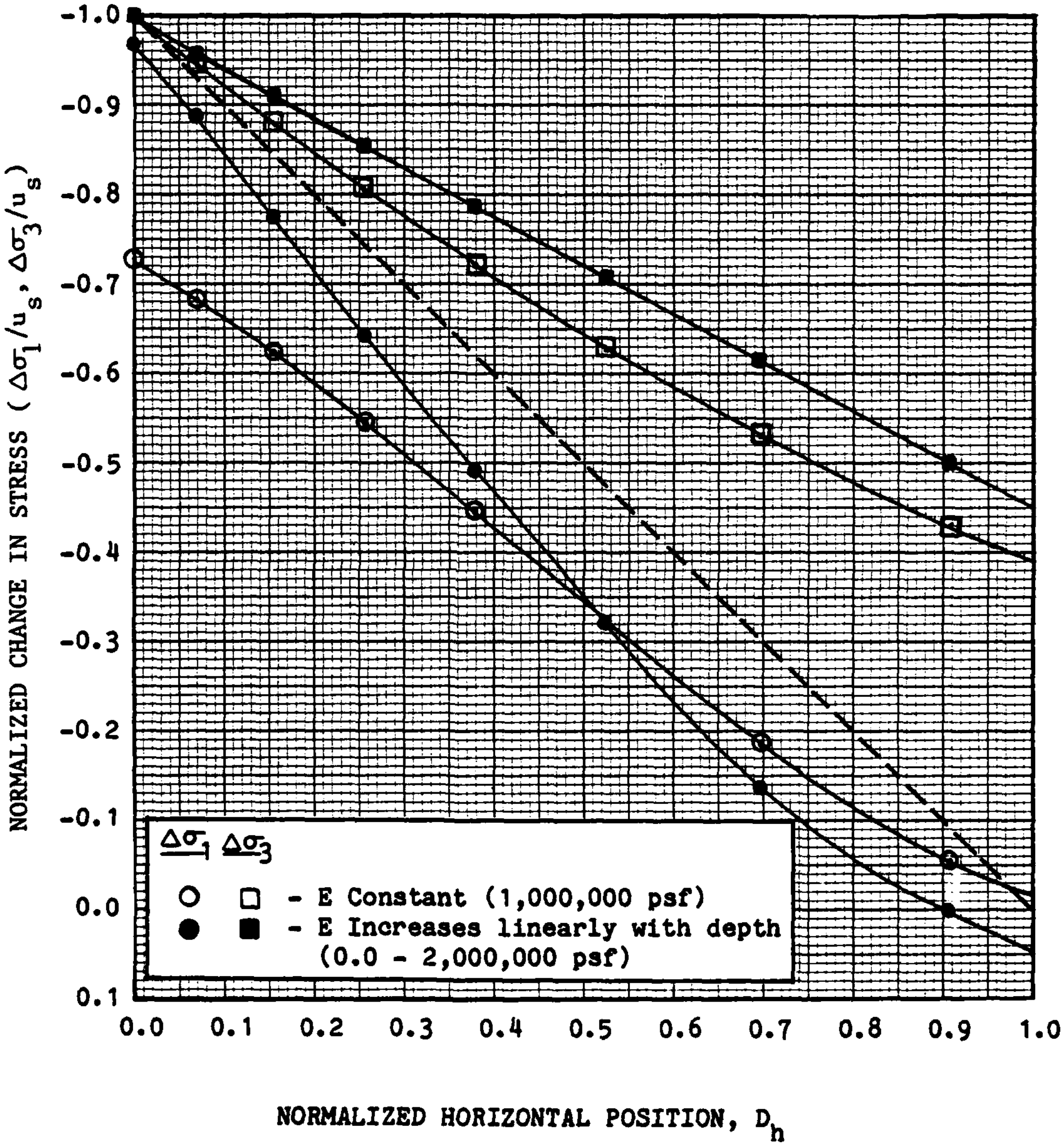


Fig. 4.4b Variation in the Major and Minor Principal Change in Stress Across a Horizontal Plane at y/h = 0.50 in a 2:1 Slope.

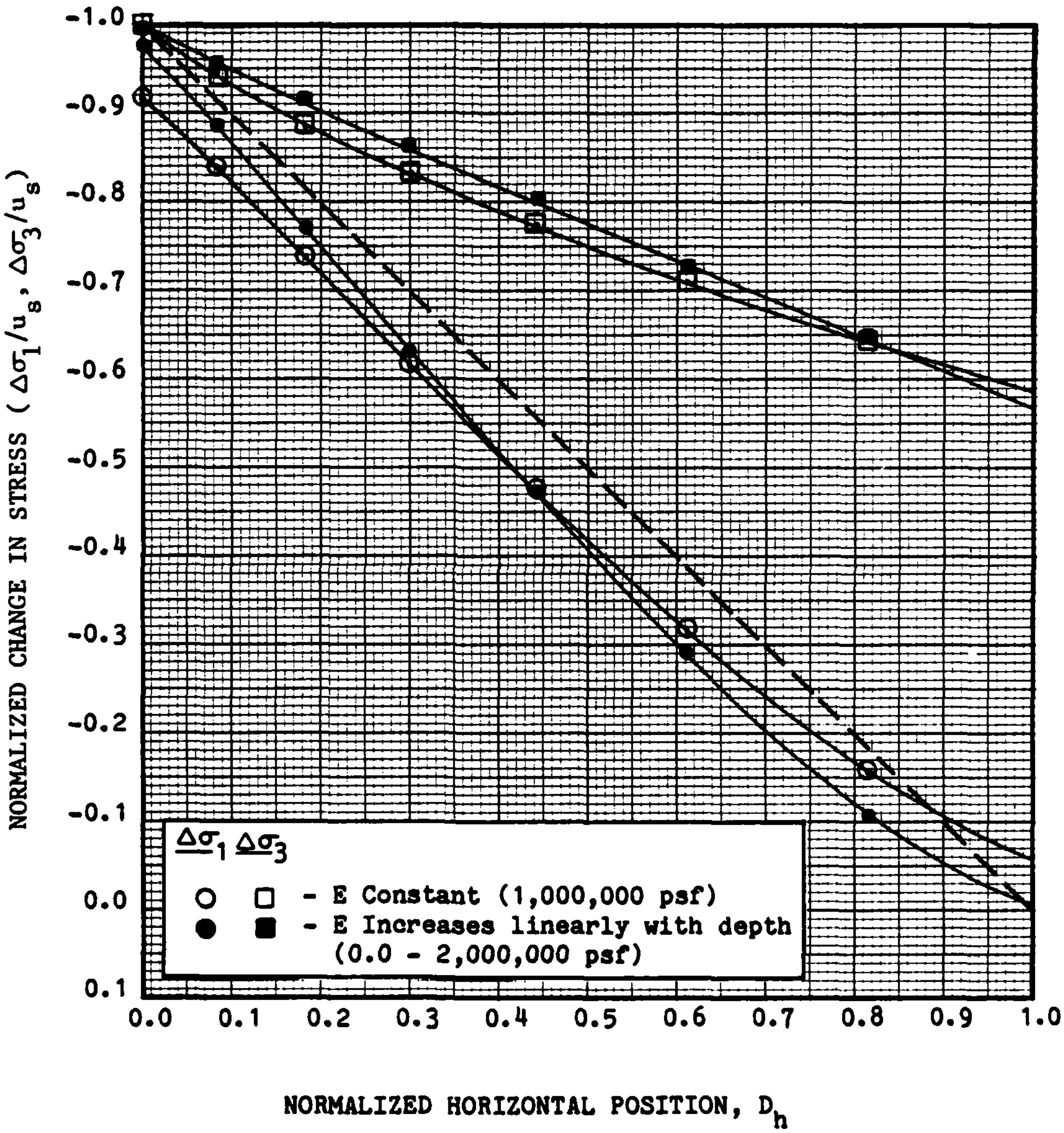


Fig. 4.4c Variation in the Major and Minor Principal Change in Stress Across a Horizontal Plane at y/h = 0.80 in a 2:1 Slope.

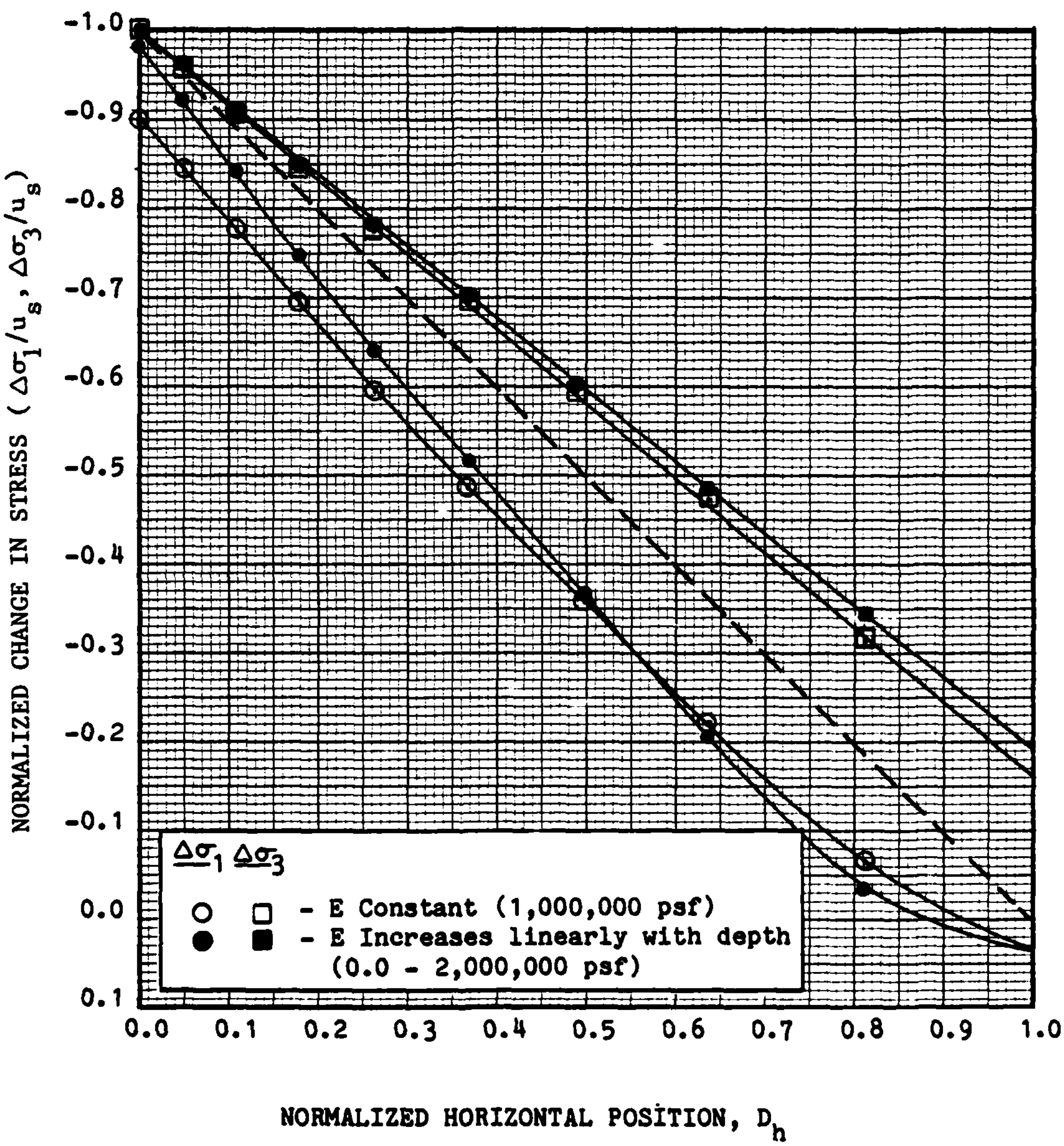


Fig. 4.5a Variation in the Major and Minor Principal Change in Stress Across a Horizontal Plane at y/h = 0.20 in a 4:1 Slope.

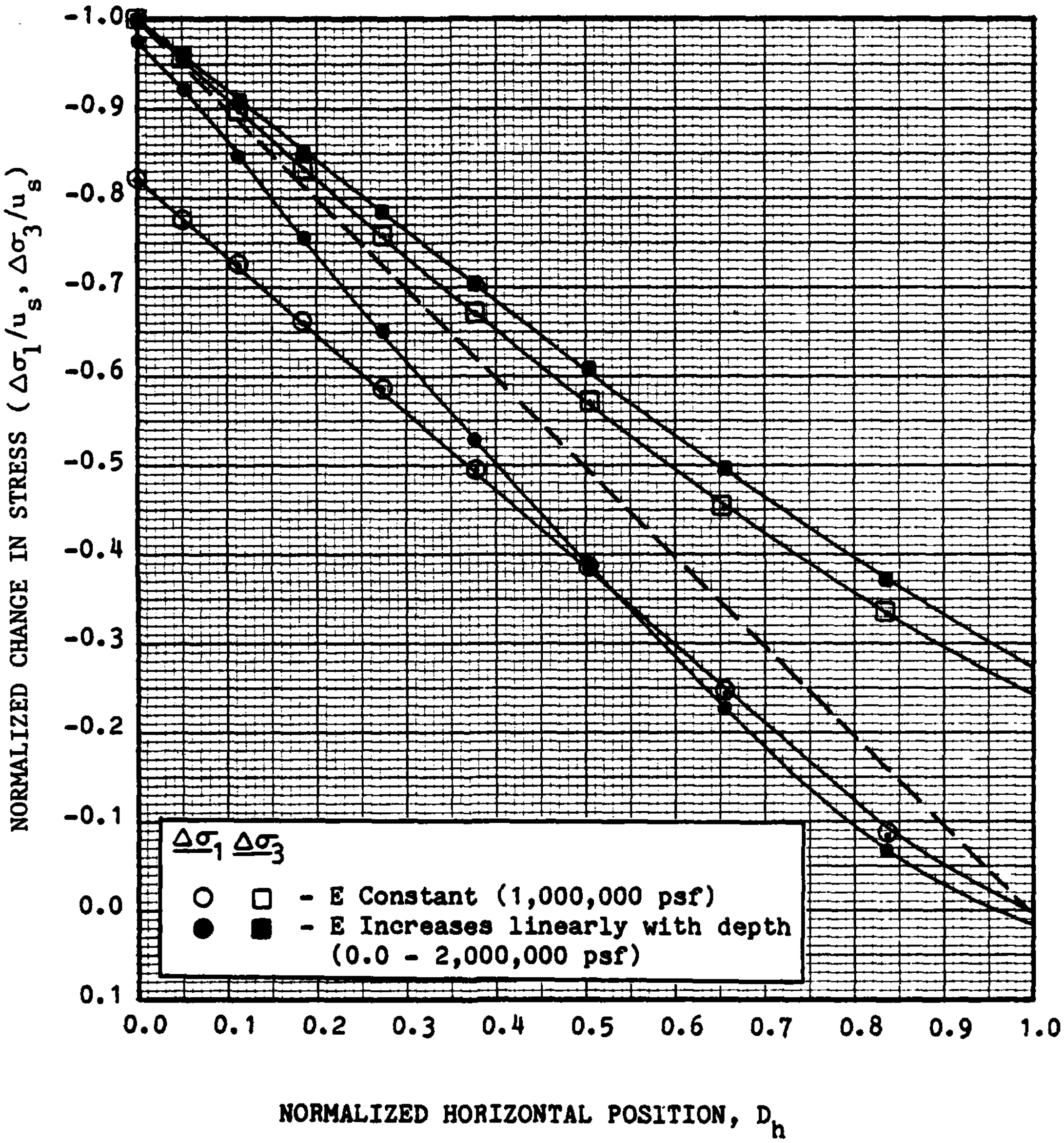


Fig. 4.5b Variation in the Major and Minor Principal Change in Stress Across a Horizontal Plane at y/h = 0.50 in a 4:1 Slope.

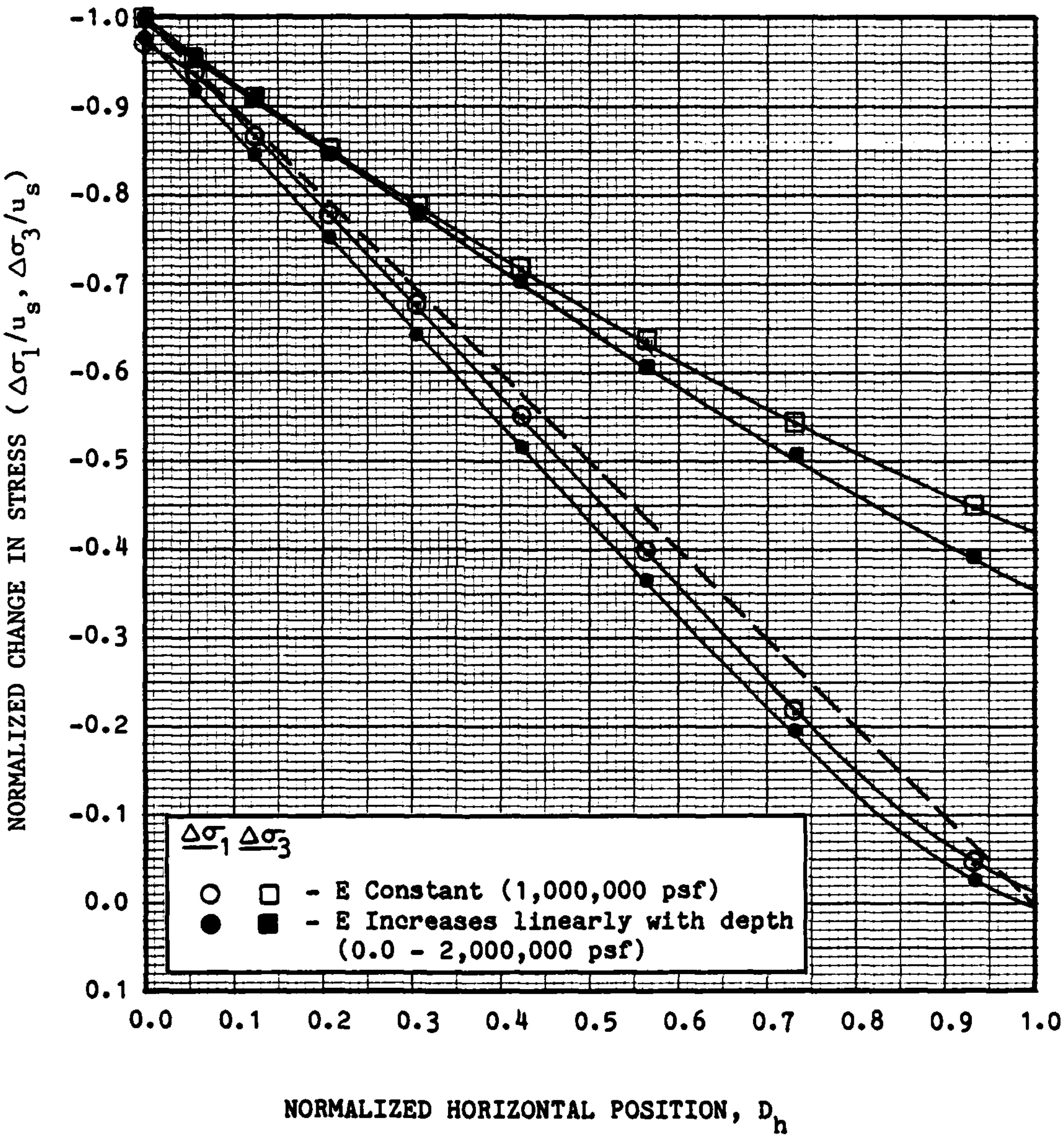


Fig. 4.5c Variation in the Major and Minor Principal Change in Stress Across a Horizontal Plane at y/h = 0.80 in a 4:1 Slope.

in stress are sensitive to the assumed variation in modulus. In the outer portion of the slope, absolute values of the major principal change in stress computed assuming a modulus which varies linearly with depth are significantly greater than those calculated assuming a constant modulus. This difference is greatest at the upstream face of the slope and can be explained by the fact that the assumption of a linearly varying modulus imposes nearly hydrostatic stress conditions at the face of the slope. Therefore, the major principal change in stress is constrained to be nearly equal to the minor principal change in stress. This finding is of considerable interest to this study because it indicates that the assumed variation in modulus will have a significant effect on values of the principal stress difference ($\Delta\sigma_1 - \Delta\sigma_3$) calculated from the results of the finite element computations.

Influence of Slope on Principal Changes in Stress.

To examine the influence of upstream slope inclination, the variation in major and minor principal change in stress across representative horizontal planes (y/h equal to .20, .50 and .80) are presented in Figures 4.6 a thru c for both a 2:1 and 4:1 upstream slope. The broken line shown in these figures represents the rapid drawdown induced reduction in hydrostatic pressure at the face of the slope immediately above the respective points on the horizontal plane. It can be seen from these figures that, as the slope becomes flatter (2:1 to 4:1), the absolute value of the major principal change in

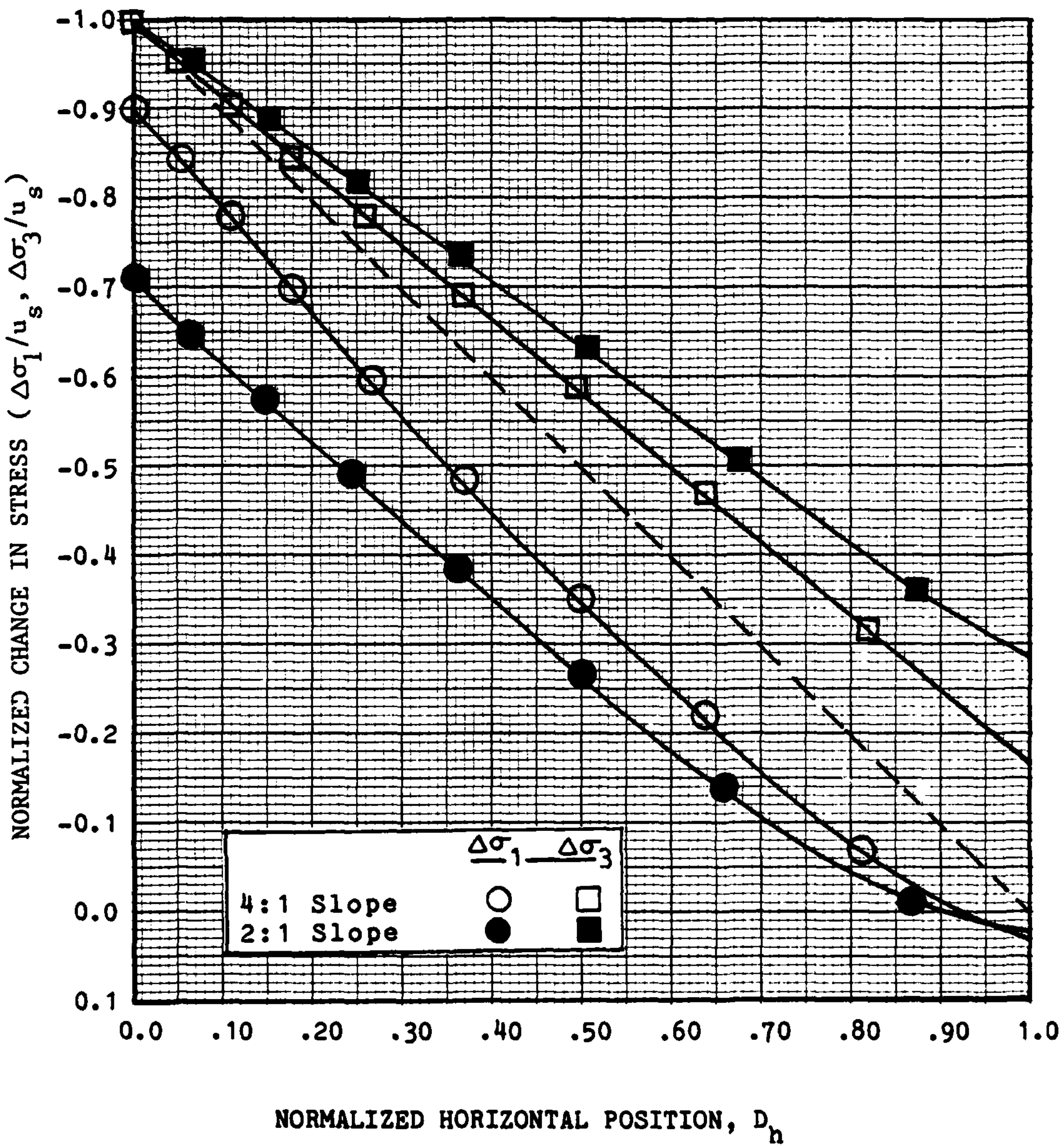


Fig. 4.6a Influence of Upstream Slope Inclination on the Variation in the Major and Minor Principal Change in Stress Across a Horizontal Plane at y/h = 0.20.

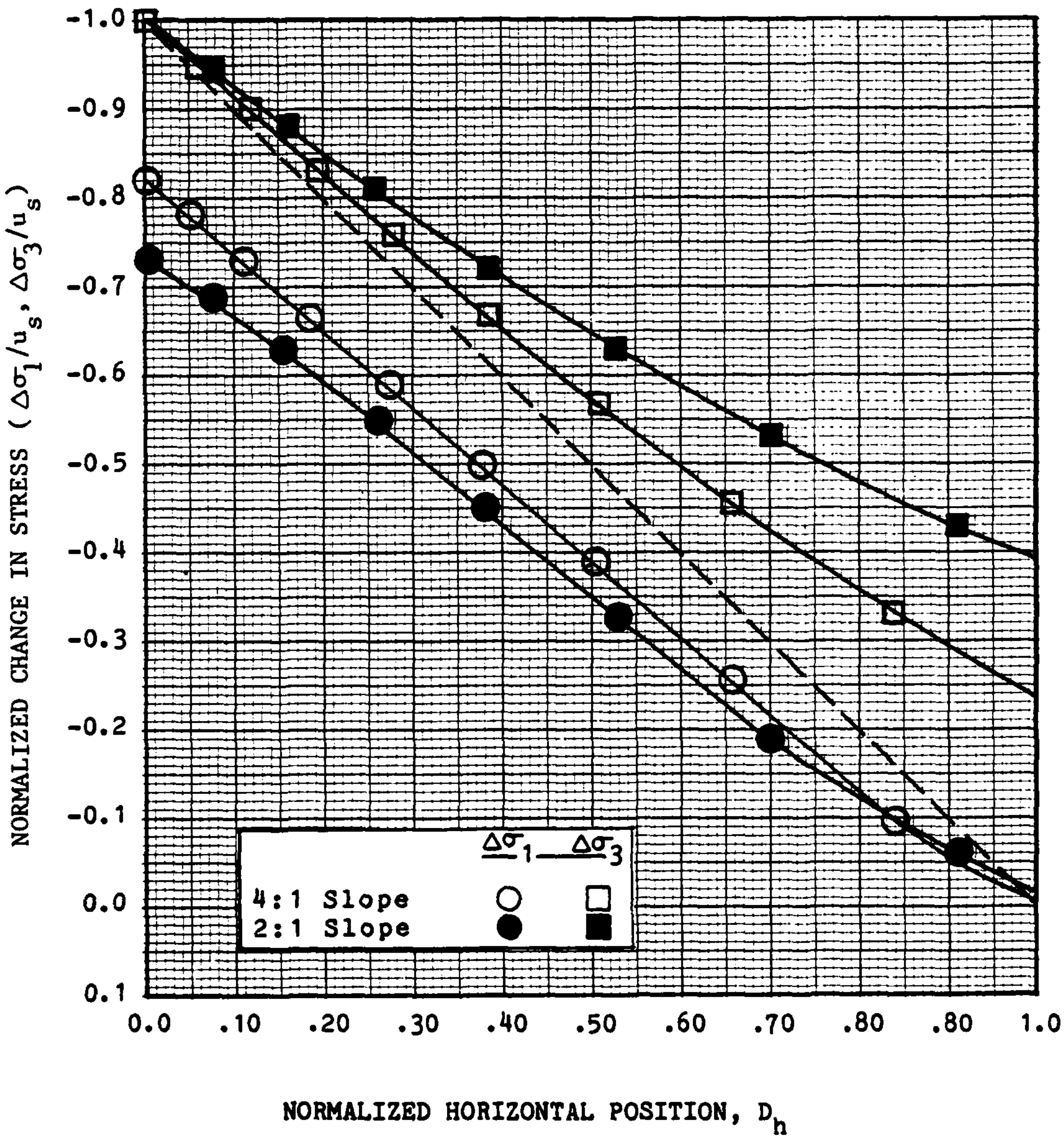


Fig. 4.6b Influence of Upstream Slope Inclination on the Variation in the Major and Minor Principal Change in Stress Across a Horizontal Plane at y/h = 0.50.

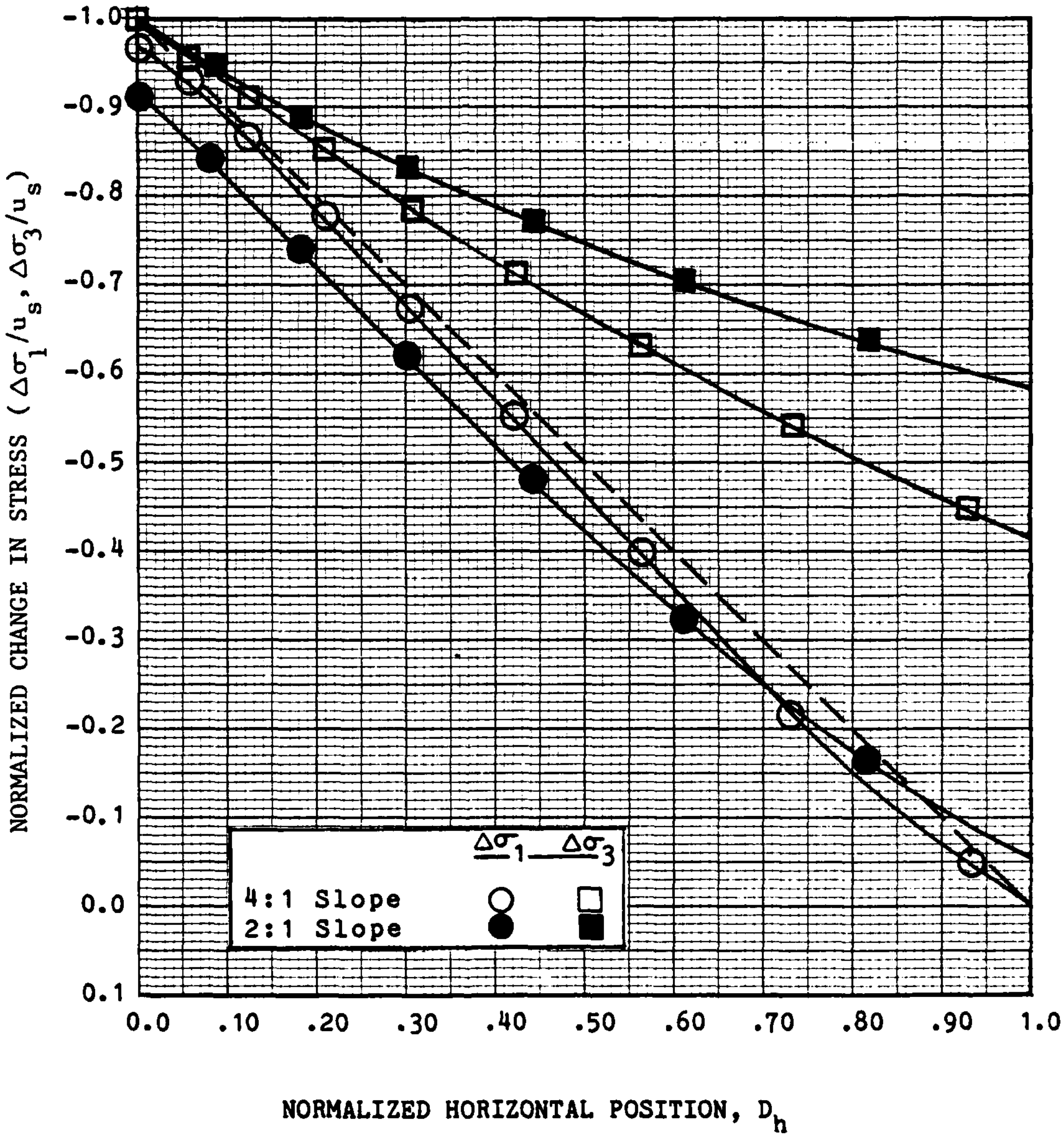


Fig. 4.6c Influence of Upstream Slope Inclination on the Variation in the Major and Minor Principal Change in Stress Across a Horizontal Plane at y/h = 0.80.

stress increases and the absolute value of the minor principal change in stress decreases at a given point within the upstream slope. This is the result of the imposed material constraint of zero volume change. As the slope becomes flatter, it is approaching the limiting condition of a horizontal surface. At the limiting condition under the constraint of zero volume change, the major and minor principal changes in stress will both be equal to the applied vertical load. This trend can be seen in Figures 4.6 a thru c where, as the slope becomes flatter, values of the major and minor principal change in stress are approaching the value of the reduction in hydrostatic pressure at the face of the slope (broken line). This finding is important to this study because it shows that slope inclination has a significant influence on values of the principal stress difference ($\Delta\sigma_1 - \Delta\sigma_3$). As the slope becomes flatter, computed values of the principal stress difference will be smaller.

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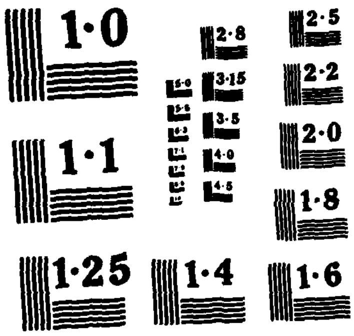

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CONCLUSIONS

Linearly elastic finite element computations were conducted to provide an estimate of the principal changes in stress which are induced in an earth slope as the result of rapid drawdown. Stress vector plots of the major and minor principal changes in stress showed that the effect of drawdown on the principal changes in stress is concentrated in the portion of the slope upstream of a vertical plane passing through the upstream crest of the dam. The assumed variation in modulus has a significant influence on the major principal change in stress. Values of the major principal change in stress computed assuming a modulus which varies linearly with depth are higher than those calculated assuming a constant modulus throughout the slope. Therefore, calculated values of the principal stress difference at points in the slope are dependent on the assumed variation in the modulus. It was also shown that the principal stress difference is effected by upstream slope inclination. As the slope becomes flatter, values of the major and minor principal change in stress are more nearly equal and the computed value of the principal stress difference will be smaller. The information obtained from the finite element computations provides a basis for evaluating the use of pore pressure coefficient equations to estimate the pore pressure changes which occur within an earth slope as the result of rapid drawdown loading.

CHAPTER 5

EVALUATION OF BISHOP'S PROCEDURE FOR ESTIMATING PORE PRESSURES DUE TO DRAWDOWN

INTRODUCTION

Bishop (1954) proposed using the pore pressure coefficient equation to estimate the changes in pore water pressure which are induced in earth slopes as the result of rapid drawdown. An investigation was conducted to evaluate the assumptions which Bishop made in developing his method. Changes in pore water pressure were calculated using the value of the pore pressure coefficient, A, assumed by Bishop and stresses calculated from the finite element method. These changes in pore pressure were compared with the values obtained using Bishop's method. Also, values of the pore pressure coefficient, which would be required to produce the changes in pore water pressure estimated using Bishop's method, were calculated using stresses from the finite element method. The pore pressure coefficients determined in that manner were compared with the value assumed by Bishop for the entire cross section of the upstream portion of a slope.

BISHOP'S METHOD

Bishop (1954) used the concept of pore pressure coefficients to develop a method to predict the pore water pressures in a earth dam after rapid drawdown. The pore pressure equation which he used is an alternate form of Skempton's (1954) equation:

$$\Delta u = B \left[\Delta\sigma_1 + A \left(\Delta\sigma_1 - \Delta\sigma_3\right)\right] \tag{5.1}$$

which is expressed in the form:

$$\frac{\Delta u}{\Delta\sigma_1} = \bar{B} = B \left[1 - (1 - A) \left(1 - \frac{\Delta\sigma_3}{\Delta\sigma_1} \right)\right] \tag{5.2}$$

where B is defined as the "overall pore pressure coefficient" by Bishop. In applying this equation to the case of rapid drawdown, Bishop made three assumptions. First, he assumed that, for fully saturated soils, the B coefficient "is almost equal to unity". Verification of this assumption is beyond the scope of this study, and therefore, for purposes of this study, the value of B is assumed to be equal to one. The second assumption made by Bishop was that, as a safe working rule, $\bar{B}$ could be taken as being equal to one. As can be seen from Equation 5.2, the value of $\bar{B}$ is dependent upon the value of the principal stress ratio, $\Delta\sigma_3/\Delta\sigma_1$, and upon the value of the pore pressure coefficient, A. In developing his procedure Bishop assumed that the value of the principal stress ratio was greater than one stating that, "the changes in total stress on drawdown are represented by a decrease in major principal stress, and, as the shear stress is increased, by an even larger drop in minor principal stress." With

the principal stress ratio having a value greater than one, it follows from Equation 5.2 that, setting the value of $\overline{B}$ equal to one implies that the value of A has also been assumed to be equal to one throughout the slope. It can also be seen from Equation 5.2 that, in setting $\overline{B}$ equal to one, Bishop constrained the change in pore pressure to be equal to the major principal change in stress. The third assumption introduced by Bishop was that the major principal change in stress could be taken as being equal to the vertical change in stress immediately above a point in the slope. For a homogeneous earth slope in which no drainage occurs during drawdown, this assumption implies that the major principal change in stress is equal to the change in hydrostatic pressure above each point in the slope. The influence which the second and third assumptions made by Bishop have on the values of the changes in pore pressure estimated using his method is evaluated in the following section.

EVALUATION OF BISHOP'S ASSUMPTIONS

Assumed Value of the Major Principal Change in Stress

Bishop (1954) assumed that, as an initial approximation, the major principal change in stress could be taken as being equal to the vertical change in stress. To study the effect of that assumption, changes in pore pressure were estimated using the pore pressure coefficient equation with an A value equal to one, as assumed by Bishop, and principal changes in stress calculated using the finite

element method. These changes in pore pressure are compared with the values assumed in Bishop's method (broken line) in Figures 5.1 a thru c for a 2:1 upstream slope and in Figures 5.2 a thru c for a 4:1 upstream slope. The variation in the change in pore pressure along representative horizontal planes through the slope at non-dimensionalized heights (y/h) of .20 (Figures 5.1a and 5.2a), .50 (Figures 5.1b and 5.2b), and .80 (Figures 5.1c and 5.2c) is shown in these figures. The changes in pore water pressure have been normalized with respect to the pore pressures on that plane prior to drawdown, u_o, assuming a horizontal water surface and no flow. The horizontal positions of data points from the upstream face of the slope are normalized with respect to the distance from the face of the slope to the crest of the dam, D_h. As can be seen from these figures, the changes in pore pressure estimated using principal changes in stress calculated from the finite element method and a pore pressure coefficient, A, equal to one are lower than those calculated using Bishop's equation. Changes in pore pressure estimated using stresses calculated from the finite element method with both a constant modulus and a modulus varying linearly with depth are presented in Figures 5.1 and 5.2. Lower changes in pore pressure, and, thus, the largest difference from the values obtained using Bishop's method occurred when the modulus was assumed to be constant throughout the slope rather than to vary linearly with depth. A comparison of the data presented in Figures 5.1 and 5.2 shows that, as the slope becomes flatter (4:1 slope), the changes in pore water pressure estimated

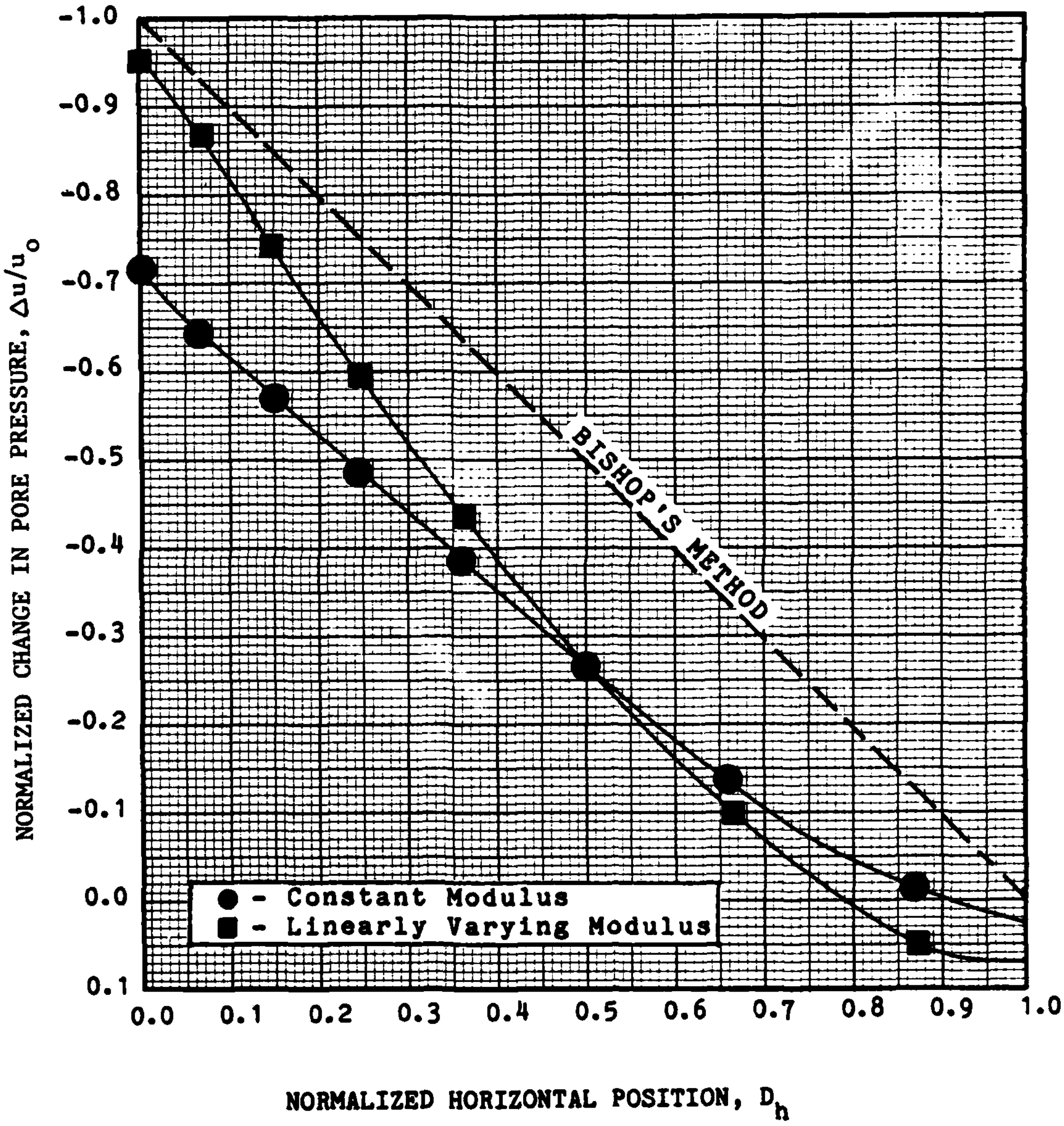


Fig. 5.1a Variation in the Change in Pore Water Pressure Along a Horizontal Plane at y/h = .20 for a 2:1 Upstream Slope.

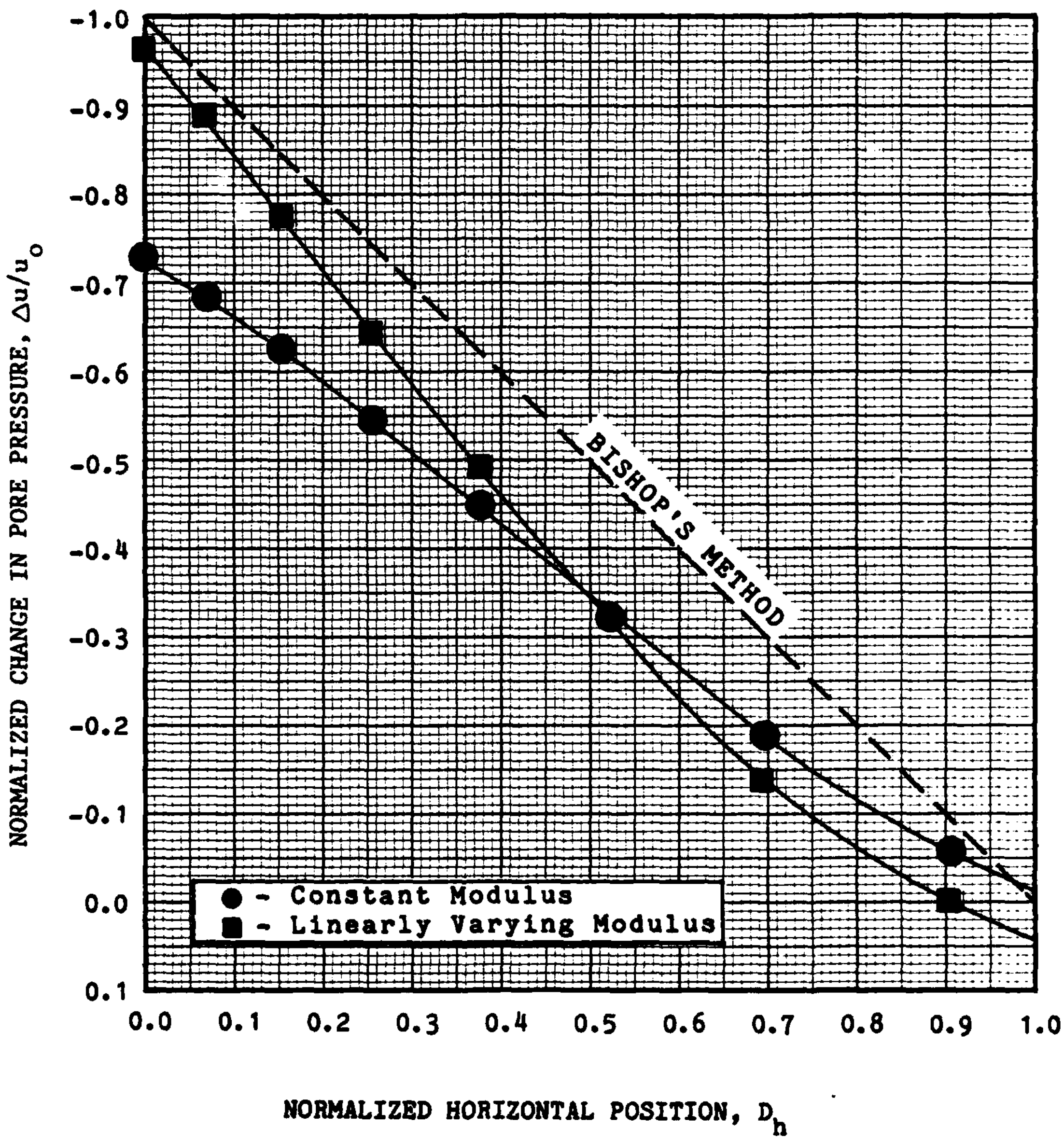


Fig. 5.1b Variation in the Change in Pore Water Pressure Along a Horizontal Plane at y/h = .50 for a 2:1 Upstream Slope.

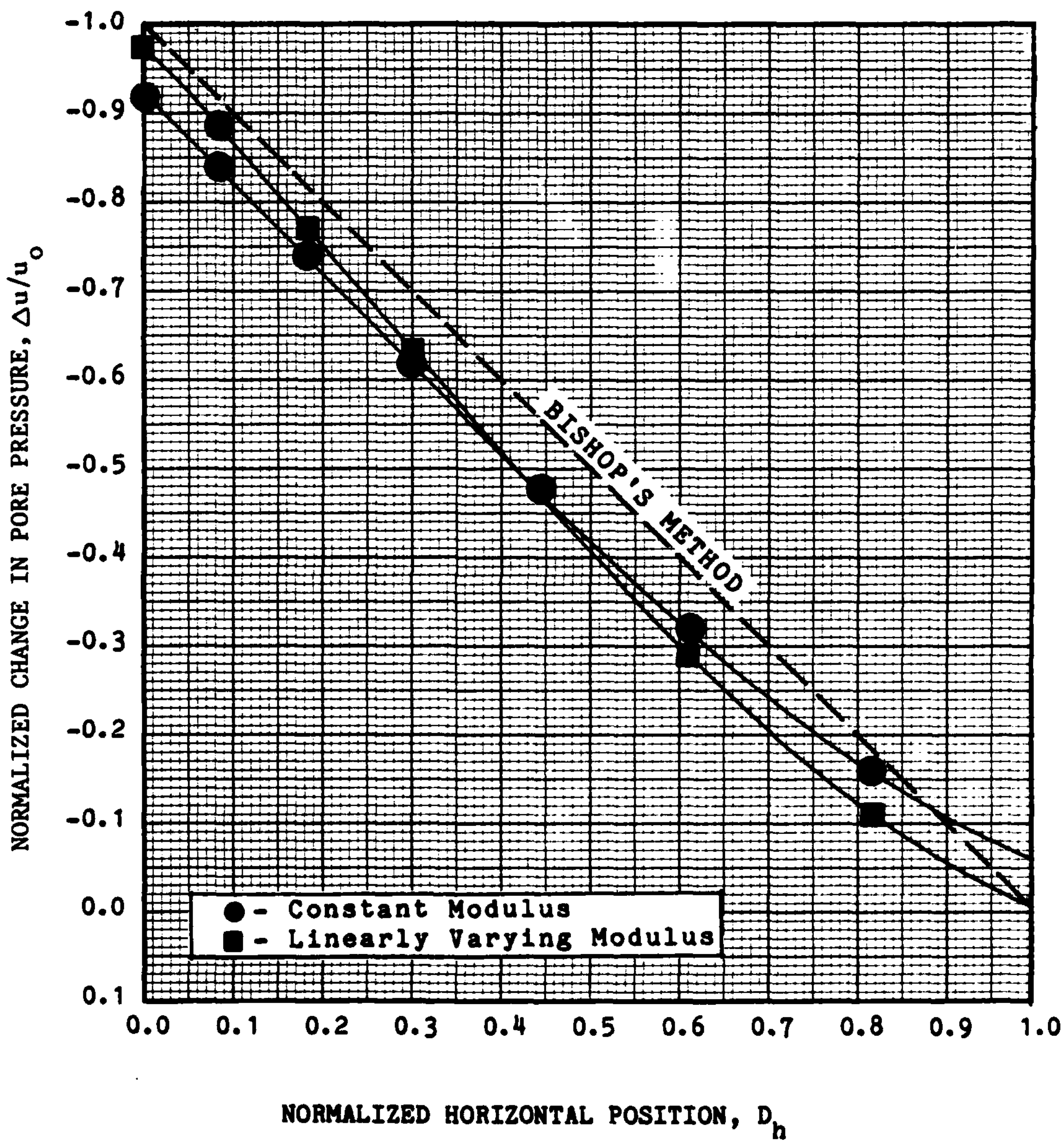


Fig. 5.1c Variation in the Change in Pore Water Pressure Along a Horizontal Plane at y/h = .80 for a 2:1 Upstream Slope.

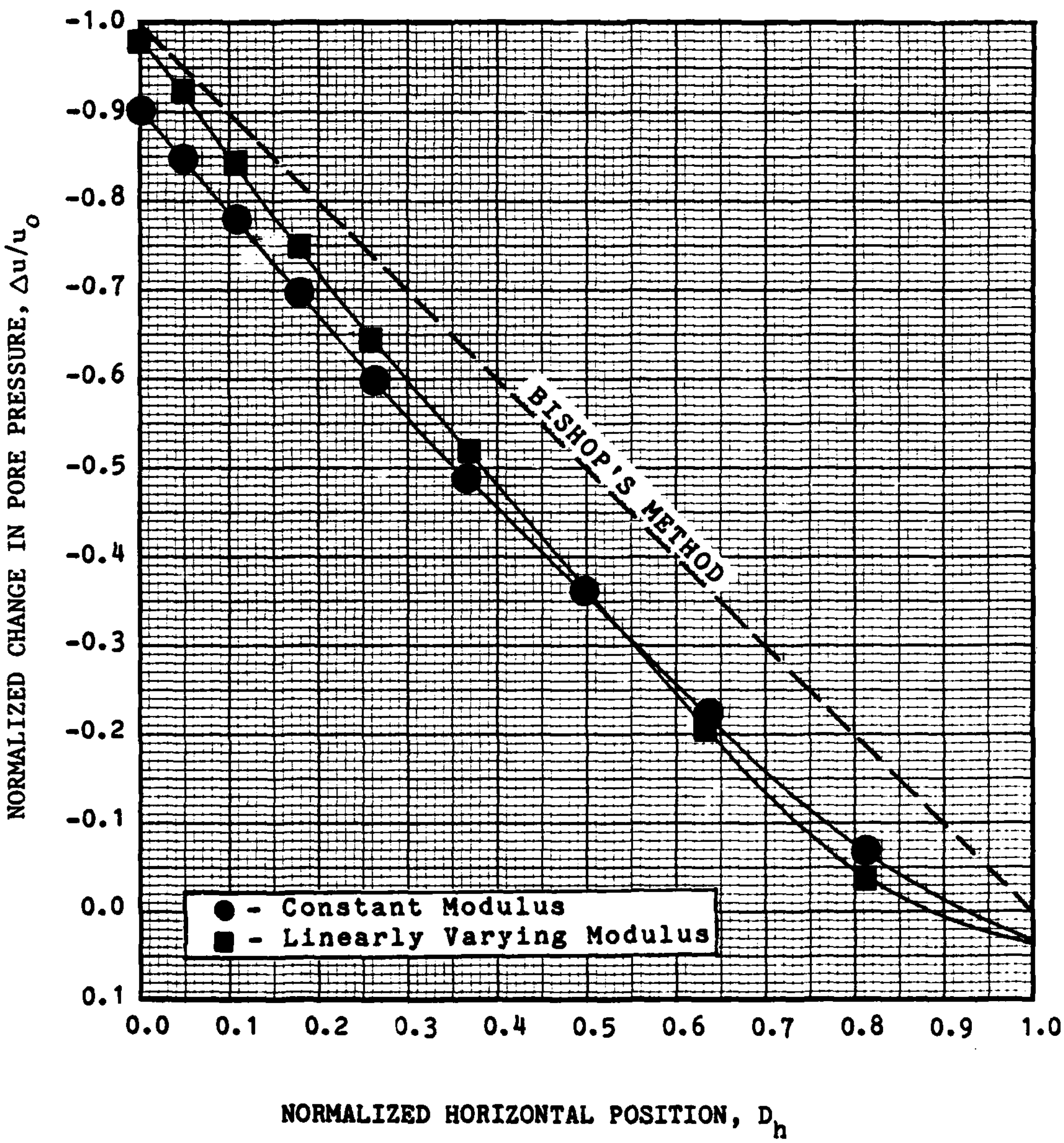


Fig. 5.2a Variation in the Change in Pore Water Pressure Along a Horizontal Plane at y/h = .20 for a 4:1 Upstream Slope.

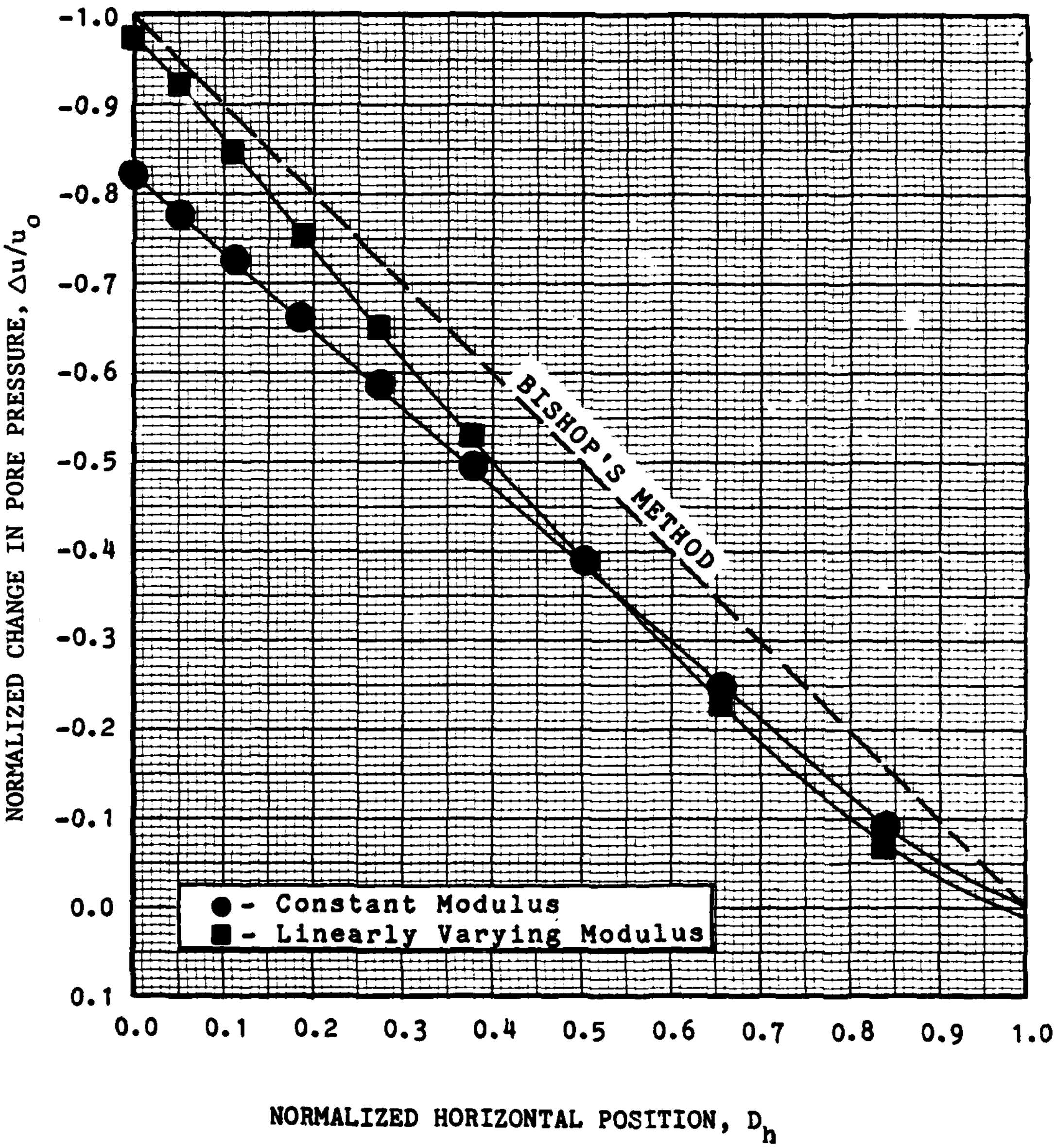


Fig. 5.2b Variation in the Change in Pore Water Pressure Along a Horizontal Plane at y/h = .50 for a 4:1 Upstream Slope.

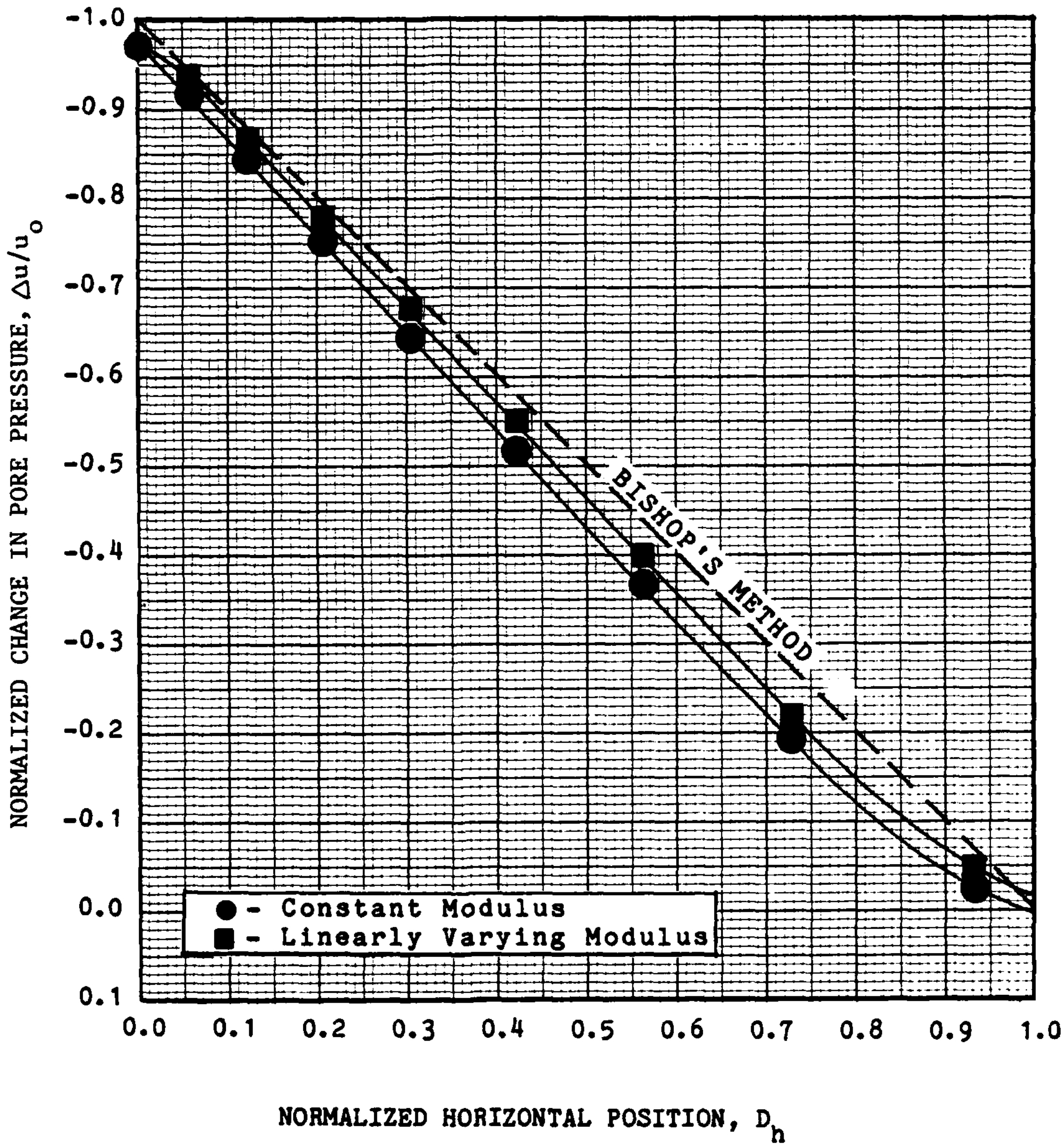


Fig. 5.2c Variation in the Change in Pore Water Pressure Along a Horizontal Plane at y/h = .80 for a 4:1 Upstream Slope.

using principal changes in stress calculated from the finite element method are more equal to the changes in pore pressure estimated using Bishop's method. This trend is the result of Bishop's assumption that the major principal change in stress at each point in the slope is equal to the vertical change in stress immediately above each point. Since this condition is exactly true for points beneath a horizontal surface, the stress state assumed by Bishop is more accurate for flatter slopes.

Comparison of Values of the Major Principal Change in Stress. An analysis of the data presented in Figures 5.1 and 5.2 showed that the changes in pore water pressure calculated using stresses from the finite element method are lower than those calculated using Bishop's procedure. Since the values of the pore pressure coefficient used to calculate the changes in pore water pressure by both procedures were identical, the differences in the estimated changes in pore pressure must be due to differences in the values of the major principal change in stress in the two methods. To examine this, the major principal changes in stress used in Bishop's method are compared with values calculated from the finite element method in Figure 5.3. This comparison shows the variation in the major principal change in stress across a representative horizontal plane at $y/h = .20$ for a 2:1 upstream slope. As can be seen, the values of the major principal change in stress used in Bishop's procedure are higher than those calculated from the finite element method. This trend is true for all horizontal planes and slope inclinations. In fact the variations seen

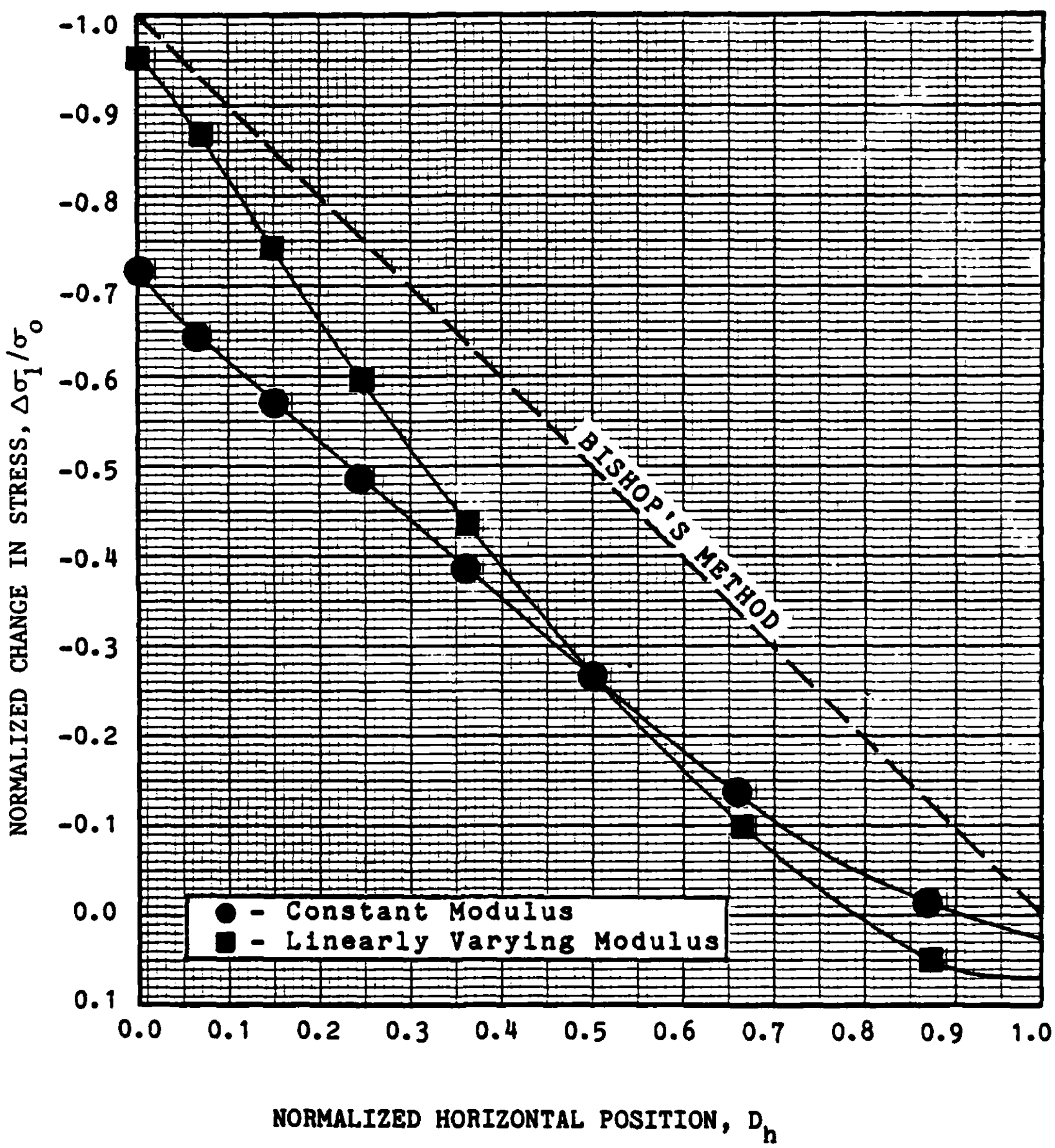
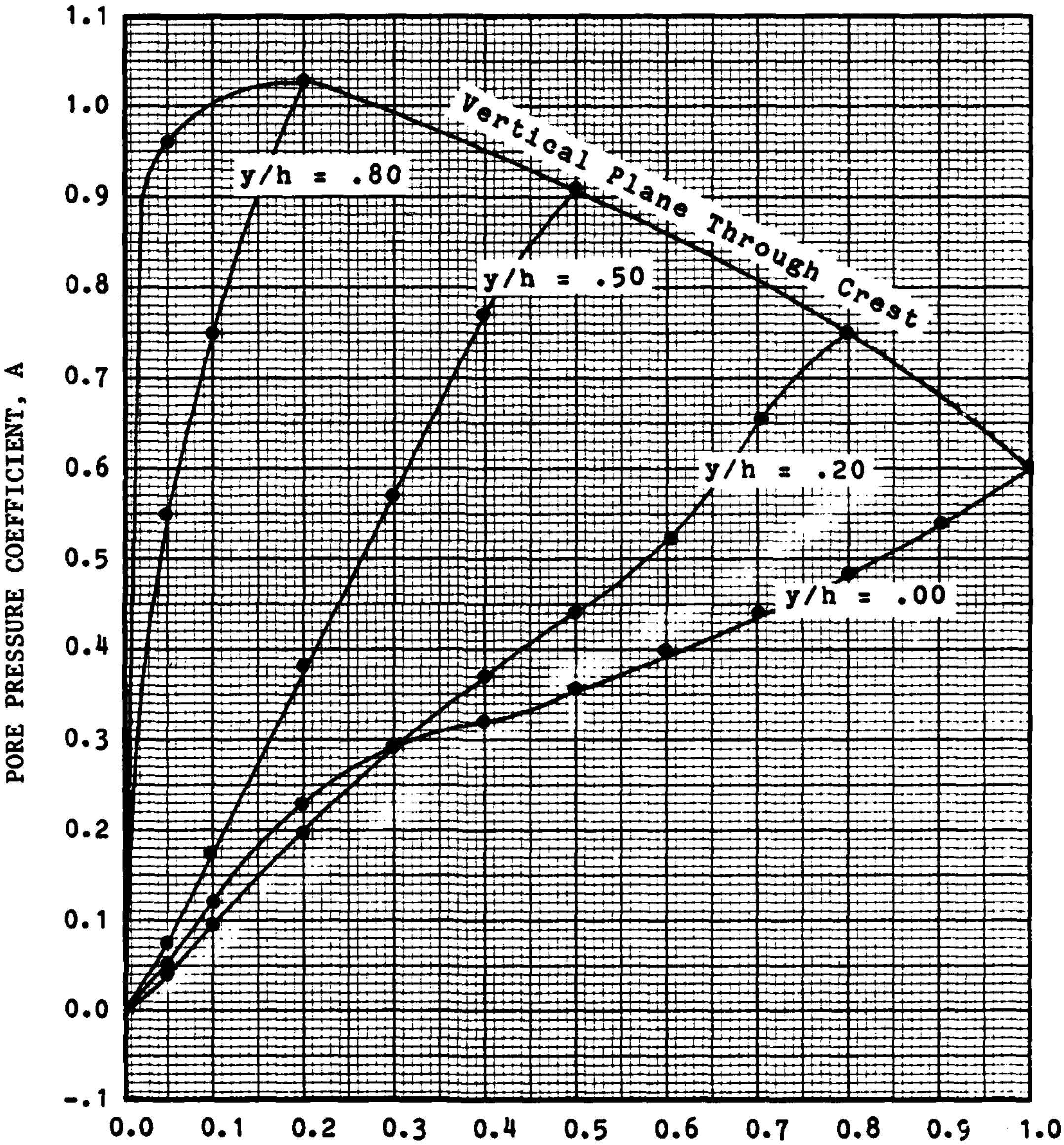


Fig. 5.3 Variation in the Major Principal Change in Stress Along a Horizontal Plane at y/h = .20 for a 2:1 Upstream Slope.

in Figure 5.3 are identical to those seen in Figure 5.1a because, with $\bar{B}$ equal to one, the change in pore water pressure is equal to the major principal change in stress. If it is assumed that a reasonable estimate of the values of the major principal change in stress is obtained from the finite element method, then the values of the major principal change in stress used in Bishop's method are overestimated.

Assumed Value of the Pore Pressure Coefficient

The value of $\bar{B}$ and, hence, the value of A are assumed to be equal to one throughout the slope in Bishop's procedure. Since this study has shown that the value of the A coefficient varies with confining pressure (Chapter 3), it seems unlikely that the pore pressure coefficient would have a constant value of one throughout the slope as assumed by Bishop. To determine how they might vary within a slope, values of the pore pressure coefficient, A, which would be required to give the changes in pore pressure estimated using Bishop's method were calculated. Principal changes in stress calculated from the finite element method were used with the pore pressure coefficient equation (Eq. 5.1) together with the values of the change in pore pressure obtained from Bishop's equation. The required values of the pore pressure coefficient, A, are plotted versus normalized vertical depth below the upstream face of the slope, D_v, in Figures 5.4 and 5.5. The variation in required values of the pore pressure coefficient along representative horizontal planes (y/h = .20, .50, and .80) are presented in these figures. Data for a 2:1 slope



NORMALIZED DEPTH WITHIN THE SLOPE, D_v

Fig. 5.4a Variation in the Value of the Pore Pressure Coefficient Required to Produce Bishop's Change in Pore Pressure with Respect to Normalized Depth in a 2:1 Slope Assuming a Constant Modulus.

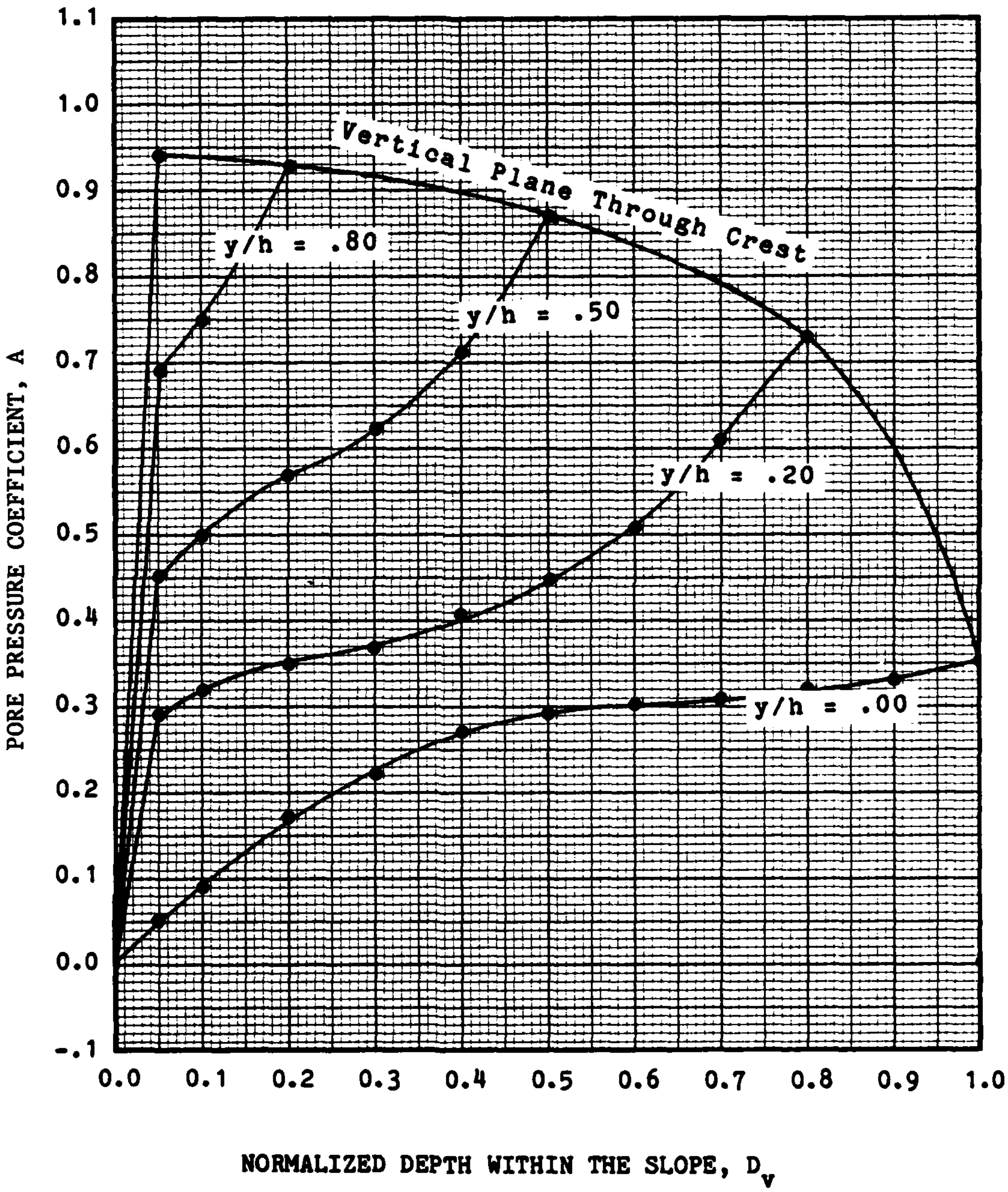


Fig. 5.4b Variation in the Value of the Pore Pressure Coefficient Required to Produce Bishop's Change in Pore Pressure with Respect to Normalized Depth in a 2:1 Slope Assuming a Linearly Varying Modulus.

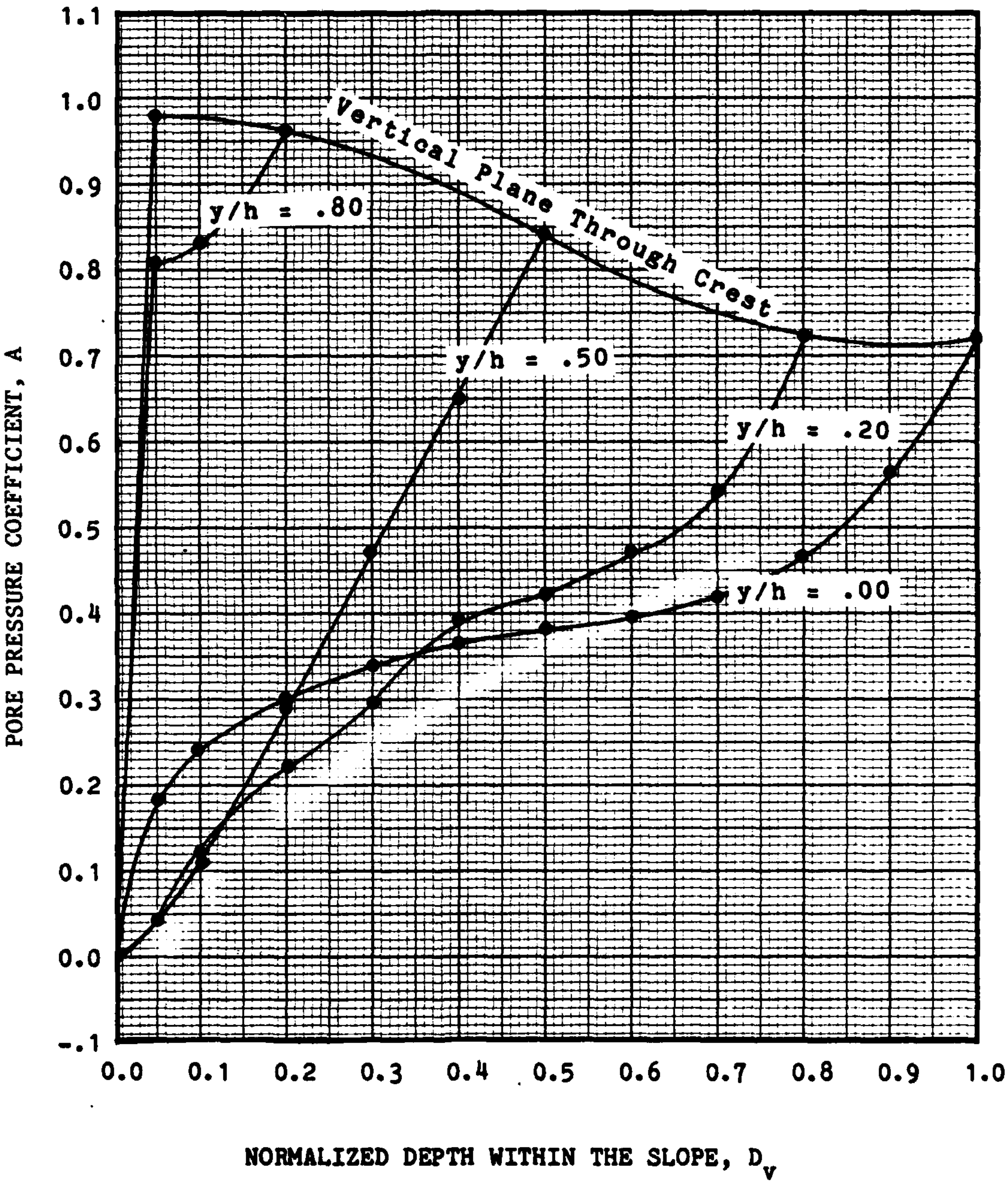


Fig. 5.5a Variation in the value of the Pore Pressure Coefficient Required to Produce Bishop's Change in Pore Pressure with Respect to Normalized Depth in a 4:1 Slope Assuming a Constant Modulus.

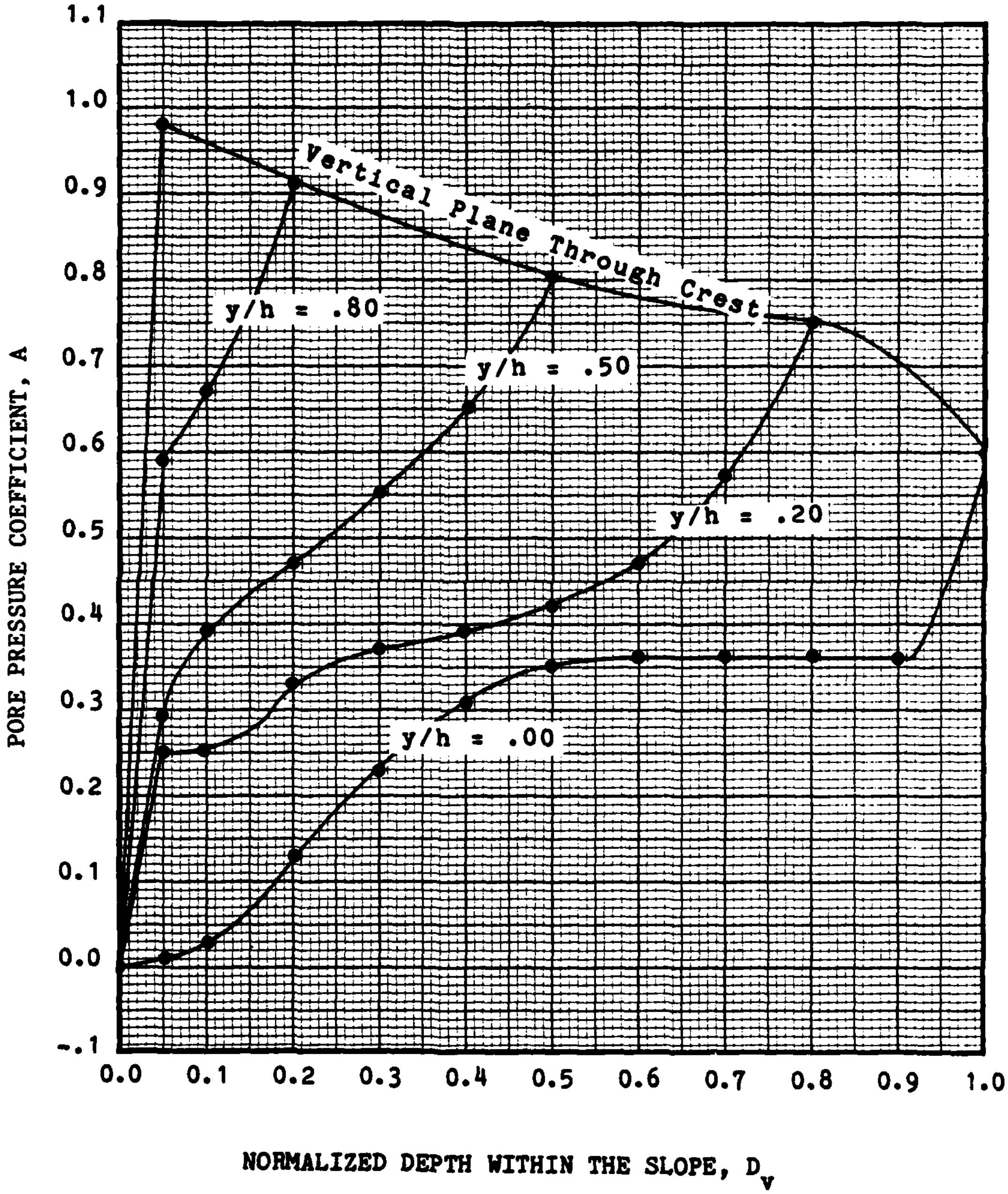


Fig. 5.5b Variation in the Value of the Pore Pressure Coefficient Required to Produce Bishop's Change in Pore Pressure with Respect to Normalized Depth in a 4:1 Slope Assuming a Linearly Varying Modulus.

are presented in Figure 5.4a for a constant modulus and, in Figure 5.4b, for a linearly varying modulus. Similar plots are presented in Figures 5.5a and b for a 4:1 upstream slope. A review of the data presented in these figures shows that the values of the pore pressure coefficient which would be required to produce the changes in pore pressure estimated using Bishop's method vary throughout the slope and are not equal to a constant value of unity as assumed by Bishop. It can be seen that Bishop has overestimated the value of the pore pressure coefficient throughout the slope.

Influence of Modulus. The influence of the assumed variation in modulus is illustrated in Figures 5.6a for a 2:1 upstream slope where values of the pore pressure coefficient which would be required to give Bishop's changes in pore pressure assuming both a constant modulus and a modulus which varies linearly with depth have been plotted together . Similarly, the influence of the assumed variation in modulus for a 4:1 upstream slope is illustrated in Figure 5.6b. The required values of A calculated assuming a constant soil modulus are shown as solid lines while those calculated assuming a linearly varying modulus are shown as broken lines. A review of the required values of A in Figures 5.6a and b shows that, in the outer portion of the slope, much lower values of the required pore pressure coefficient are indicated if the soil modulus is assumed to be constant throughout the slope.

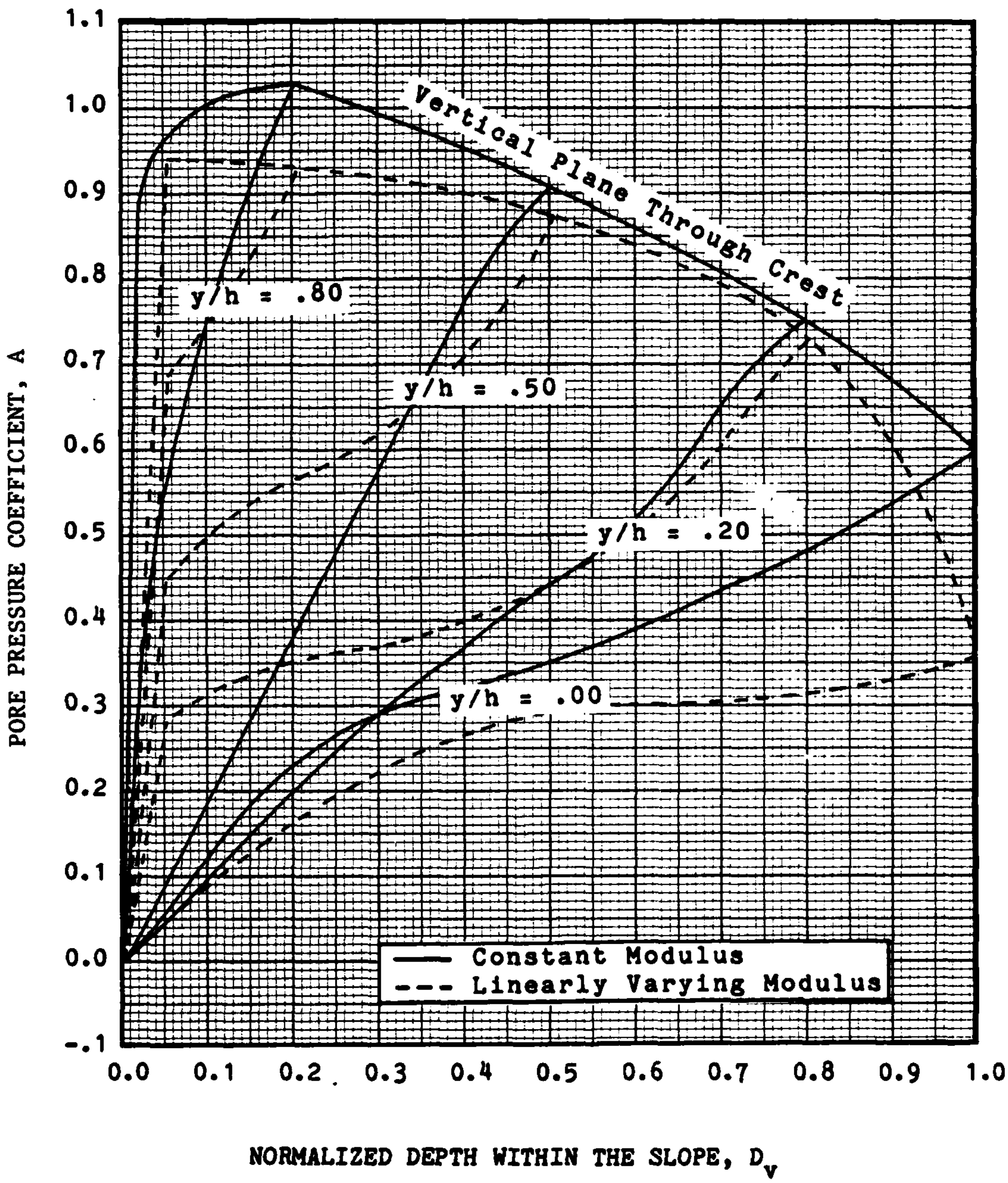


Fig. 5.6a Variation in the Value of the Pore Pressure Coefficient Required to Produce Bishop's Change in Pore Pressure with Respect to Normalized Depth in a 2:1 Slope Assuming Both a Constant and a Linearly Varying Modulus.

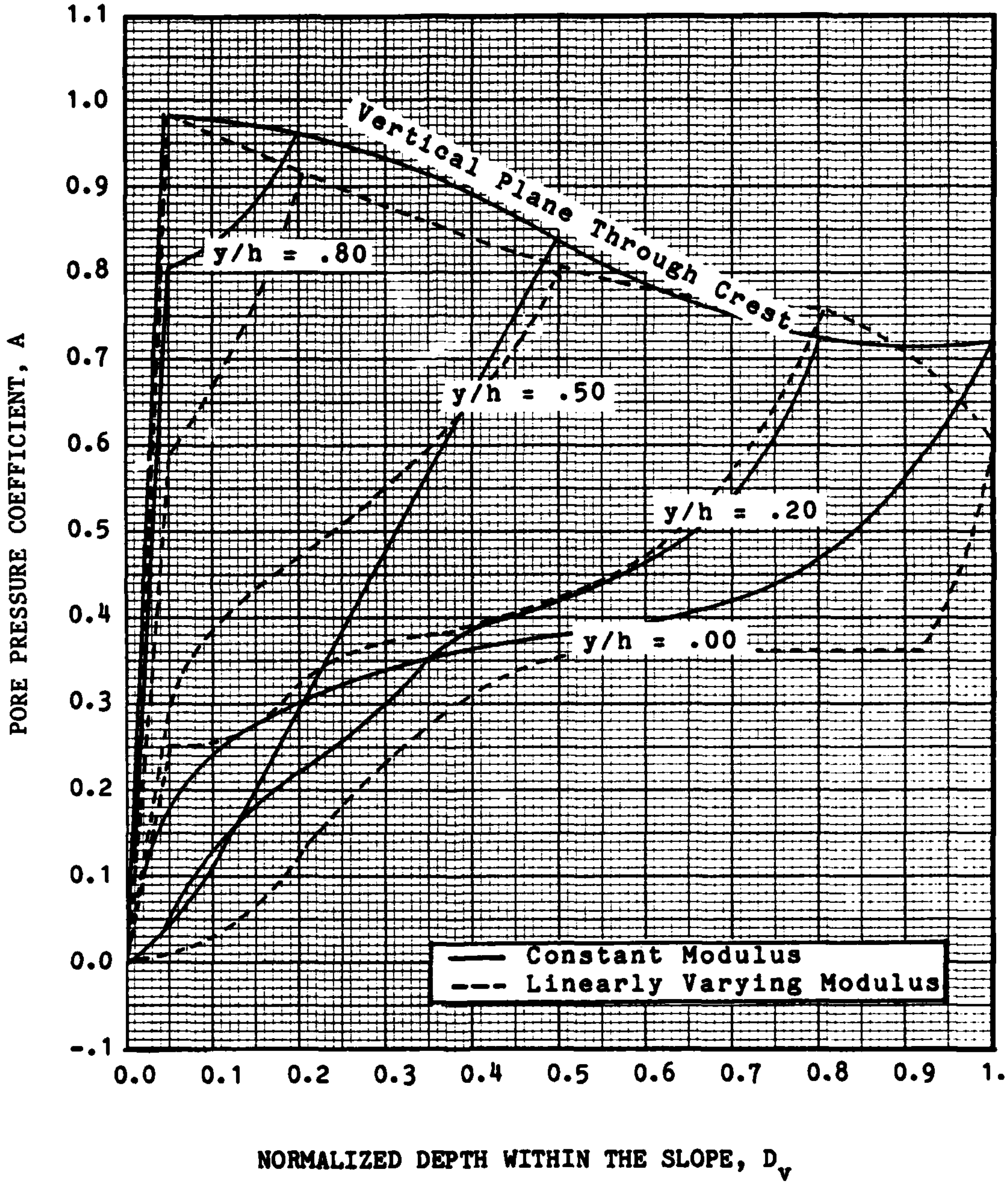


Fig. 5.6b Variation in the Value of the Pore Pressure Coefficient Required to Produce Bishop's Change in Pore Pressure with Respect to Normalized Depth in a 4:1 Slope Assuming Both a Constant and a Linearly Varying Modulus.

CONCLUSIONS

In comparisons conducted using stresses calculated from the finite element method, it has been shown that the changes in pore water pressure calculated using Bishop's method are overestimated. The principal changes in stress assumed in Bishop's procedure are higher than those calculated from a linear elastic finite element analysis. In addition, it was shown that it is unlikely that the pore pressure coefficient, A, would have a constant value of one throughout the slope as assumed by Bishop. The evaluations conducted in this study show that the value of the pore pressure coefficient which would be required to produce the changes in pore pressure calculated using Bishop's procedure vary throughout the slope.

CHAPTER 6

USE OF MEASURED PORE PRESSURE COEFFICIENTS TO PREDICT PORE PRESSURES DUE TO DRAWDOWN

INTRODUCTION

It has been shown that changes in pore water pressure calculated using finite element stresses are lower than the changes in pore pressure calculated using Bishop's method. However, reports in the literature (Morgenstern (1963), Paton and Semple (1960), Nonveiller (1957), Desai (1977) and Winkley (1982)) indicate that reasonable estimates of the changes in pore pressure induced by rapid drawdown are obtained using Bishop's method. This suggests that Bishop's assumptions of a constant value for the pore pressure coefficient and his assumed values for the major principal change in stress may be compensated for by the magnitude and variation in actual values of the pore pressure coefficient for a given soil. To investigate this possibility, a literature search was conducted to obtain measured values of the pore pressure coefficient for compacted soils used in earth dam construction. The data collected from the literature are summarized in Table 6.1. To relate values of the pore pressure coefficient measured at different confining pressures to vertical depth within a slope, the confining pressure, σ_c', was divided

Table 6.1 Summary of Pore Pressure Coefficient Comparison Data for Compacted Clays

SOIL	CE[1]	CMC[2]	σ_c'	h	D_v	A	REFERENCE
Binga Dam	MA	W	864	150'	.08	.13	Lowe & Karafiath
	"	"	2592	"	.25	.28	(1960)
	"	"	6480	"	.62	.63	
Canyon Dam	HM	D	4176	250'	.24	.26	Casagrande &
	"	O	16416	"	.94	.84	Hirschfeld (1960)
Oroville	DW	O	8640	770'	.16	.69	Hall & Gordon (1966)
	"	"	18000	"	.33	.85	
	"	"	36000	"	.67	1.01	
	"	"	61200	"	1.13	1.10	
	"	"	93600	"	1.77	1.05	
St.Croix	SP	D	1440	100'	.20	.24	Johnson & Lovell
	"	"	2880	"	.41	.50	(1979)
	"	"	5760	"	.82	.53	
	"	O	1440	"	.20	-.15	
	"	"	2880	"	.41	-.01	
	"	"	4990	"	.71	.18	
	"	"	5760	"	.82	.24	
	"	W	1440	"	.20	.16	
	"	"	2880	"	.41	.29	
	"	"	5760	"	.82	.39	
	MP	W	1587	100'	.23	.00	
	"	"	1774	"	.25	.11	
	"	"	2880	"	.41	-.08	
	"	"	4322	"	.62	.06	
	"	"	5760	"	.82	-.02	

1. Compactive Effort (MA - Modified AASHTO, HM - Harvard Miniature, DW - Department of Water Resources, SP - Standard Proctor, MP - Modified Proctor)
2. Compaction Moisture Content (W - Wet of Optimum, O - At Optimum, D - Dry of Optimum)

by an assumed buoyant unit weight, γ', of 70 pounds per cubic foot. These values were then normalized by dividing them by an assumed slope height, h, giving the values of D_v presented in Table 6.1. It should be noted that, in selecting values for h, no attempt was made to model actual slope heights. Rather, to take full advantage of all the data presented by the authors, the height of the slope was selected to correspond approximately to the highest value of confining pressure used in each test series. The influence which the assumed value of slope height has on the relation between measured and required values of A will be discussed in a following section.

Comparisons of Calculated Changes in Pore Water Pressure with Changes in Pore Water Pressure Computed Using Bishop's Method.

Changes in pore water pressure were calculated with Skempton's pore pressure equation using finite element stresses and measured pore pressure coefficients for Binga Dam Clay, Canyon Dam Clay, Oroville Dam Clay, and Saint Croix Clay. These calculated changes in pore pressure were compared with the changes in pore pressure computed using Bishop's procedure as presented in the following sections. In all cases, the finite element stresses were computed assuming a constant modulus because, as shown in Chapter 5, they give the lowest and, hence, more conservative values of the changes in pore pressure.

Binga Dam Clay. Lowe and Karafiath (1960) conducted triaxial compression tests on specimens of compacted clay used in the core of the Binga Dam in the Phillippines. The specimens were prepared by

compacting samples of the clay using modified AASHTO compaction criteria at a moisture content wet of optimum. Changes in pore pressure were calculated using measured values of the pore pressure coefficient for Binga Dam Clay and principal changes in stress determined from the finite element method assuming a constant modulus. These values are compared with the changes in pore pressure calculated using Bishop's method in Figure 6.1 assuming a dam height of 150 feet. As can be seen, the changes in pore pressure on a plane at y/h = .20 calculated using Bishop's method are higher than those calculated using finite element stresses. However, as vertical position in the slope increases (y/h = .20 to y/h = .80), the changes in pore pressure computed using Bishop's procedure are increasingly lower than those calculated using finite element stresses. In addition, it can be seen that, as horizontal position along a given plane moves away from the face of the slope (D_h = 0.0), the changes in pore pressure computed using Bishop's procedure are increasingly lower than those calculated using finite element stresses.

Canyon Dam Clay. The results of a series of triaxial compression tests conducted on specimens of compacted clay used in the Canyon Dam Project in Texas were reported by Casagrande and Hirschfeld (1960). The specimens were prepared from samples of the clay which were compacted using a Harvard miniature compaction device at the optimum moisture content. Changes in pore pressure calculated using measured values of the pore pressure coefficient are compared with

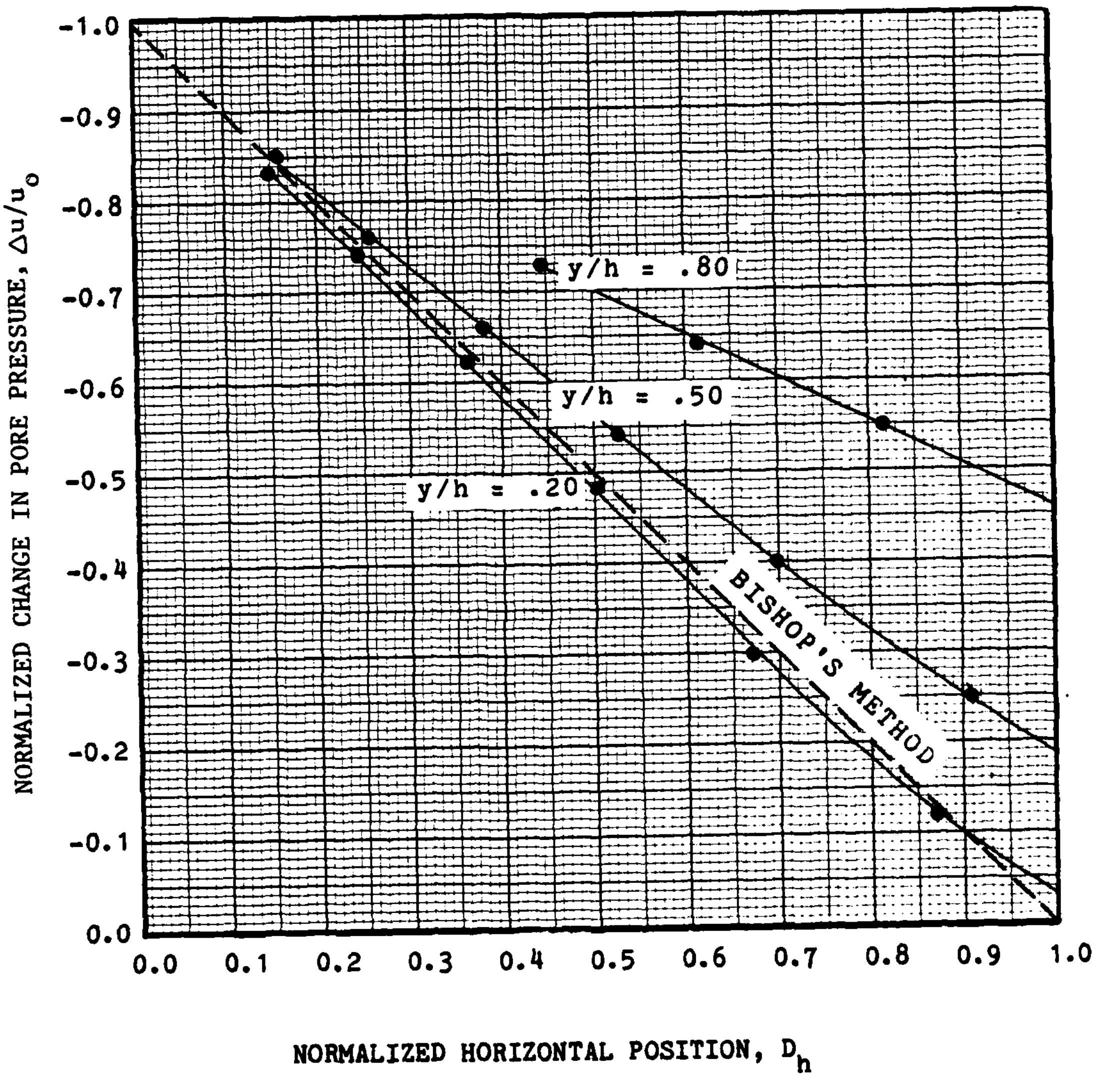


Fig. 6.1 Variation in the Change in Pore Water Pressure Calculated Using Measured Values of the Pore Pressure Coefficient for Binga Dam Clay Along Representative Horizontal Planes in a 2:1 Slope 150 Feet High.

values obtained using Bishop's method in Figure 6.2 assuming a slope height of 250 feet. An analysis of the data presented in this figure shows that changes in pore pressure computed using finite element stresses will be lower in the bottom 20 percent of the slope and higher in the upper 80 percent of the slope than those calculated using Bishop's method.

Oroville Dam Clay. The results of triaxial compression tests conducted on specimens of a compacted clay used as the impervious fill for Oroville Dam, California were reported by Hall and Gordon (1966). The test specimens were prepared from samples of the soil which were compacted to 95 percent of the maximum dry unit weight as defined by Department of Water Resources compaction criteria at the optimum moisture content. These triaxial tests were conducted at extremely high initial confining pressures which ranged from a low value of 8640 psf (60 psi) to a high value of 93,600 psf (650 psi). A slope height of 770 feet which is the actual height of Oroville Dam was used. Values of the change in pore pressure calculated using measured values of the pore pressure coefficient and finite element stresses are compared with the values calculated using Bishop's method in Figure 6.3. It is apparent from the relationships shown in this figure that, in the lower half of the slope, the changes in pore pressure computed using finite element stresses are significantly lower than those which would be calculated using Bishop's method.

Saint Croix Clay. Johnson and Lovell (1979) conducted an extensive series of triaxial compression tests on specimens of

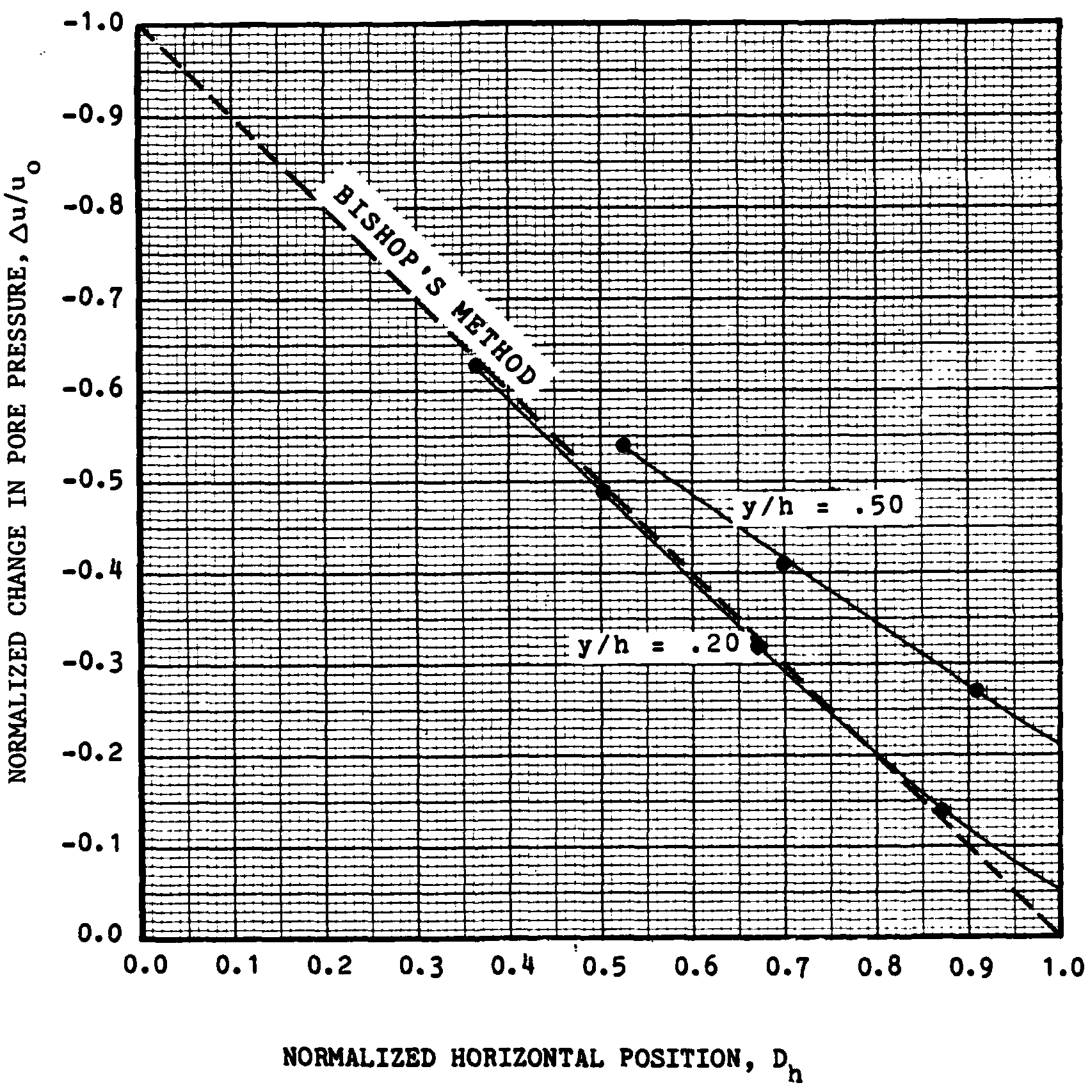


Fig. 6.2 Variation in the Change in Pore Water Pressure Calculated Using Measured Values of the Pore Pressure Coefficient for Canyon Dam Clay Along Representative Horizontal Planes in a 2:1 Slope 250 Feet High.

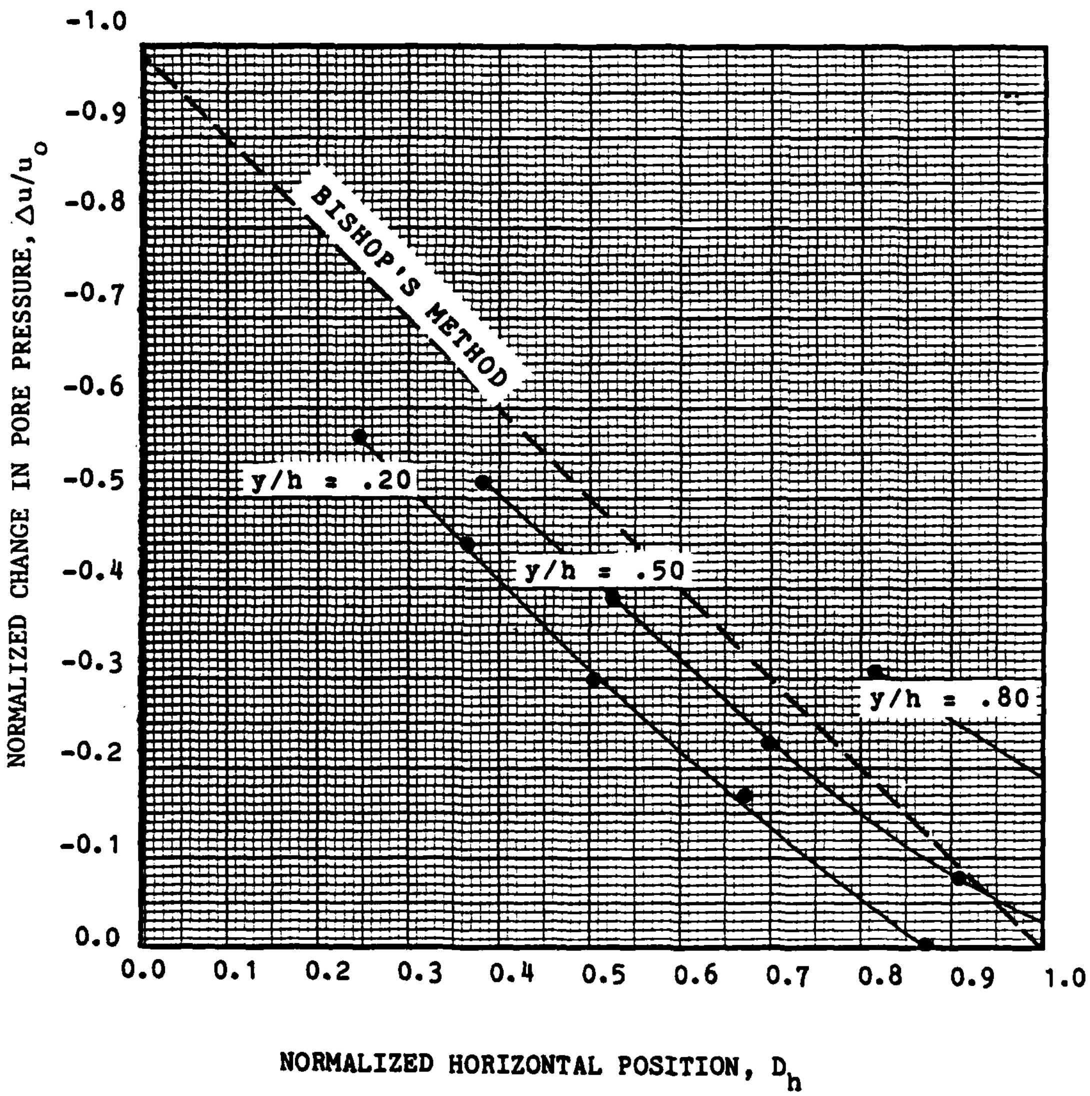


Fig. 6.3 Variation in the Change in Pore Water Pressure Calculated Using Measured Values of the Pore Pressure Coefficient for Oroville Dam Clay Along Representative Horizontal Planes in a 2:1 Slope 770 Feet High.

compacted Saint Croix clay. The test specimens were prepared from samples of the clay which were prepared using both Standard Proctor and Modified Proctor compaction criteria at water contents which were dry and wet of optimum and at the optimum moisture content. A slope height of 100 feet was assumed. The changes in pore pressure calculated using finite element stresses and measured values of A obtained from tests conducted on Standard Proctor specimens compacted at the optimum moisture content are compared with values obtained from Bishop's method in Figure 6.4. An evaluation of this figure shows trends which are identical to those found for Binga Dam Clay earlier. The changes in pore pressure computed using finite element stresses are increasingly higher than those calculated using Bishop's method at points in the slope above the base and behind the upstream face of the slope. To study the influence of compaction moisture content, changes in pore pressure were calculated using measured values of A for specimens of Saint Croix clay compacted at moisture contents wet and dry of optimum and at the optimum moisture content using Standard Proctor compaction criteria. These values are compared with changes in pore pressure calculated using Bishop's procedure in Figure 6.5. A representative horizontal plane at $y/h = .20$ and an assumed slope height of 100 feet was used. The compaction moisture content had a small influence on the calculated values of the change in pore pressure as shown in this figure. For Saint Croix clay, the greatest difference between changes in pore pressure computed using Bishop's method and those calculated using finite element stresses occurs when

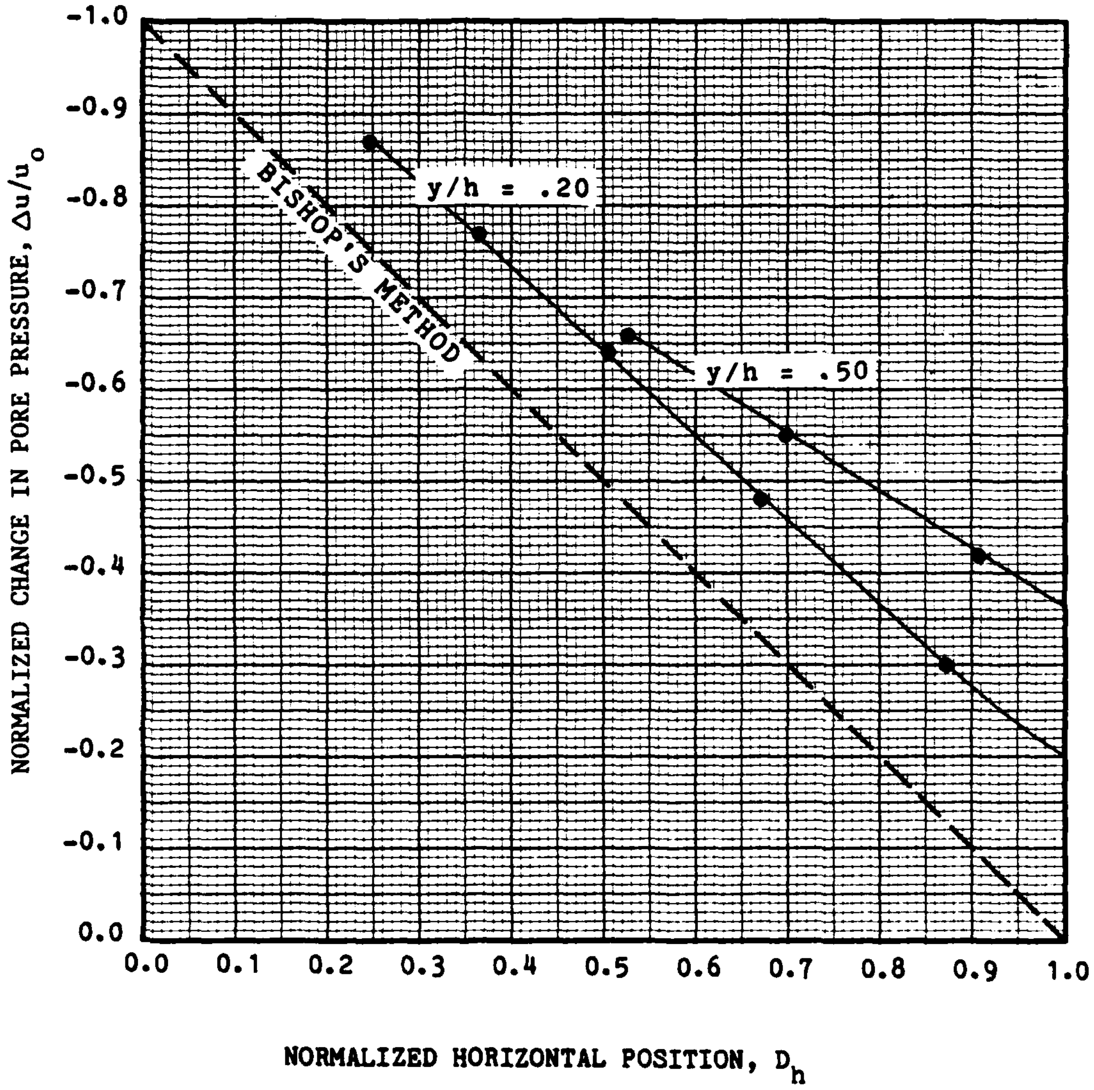


Fig. 6.4 Variation in the Change in Pore Water Pressure Calculated Using Measured Values of the Pore Pressure Coefficient for Saint Croix Clay Compacted at OMC Using Standard Proctor Criteria Along Representative Horizontal Planes in a 2:1 Slope 100 Feet High.

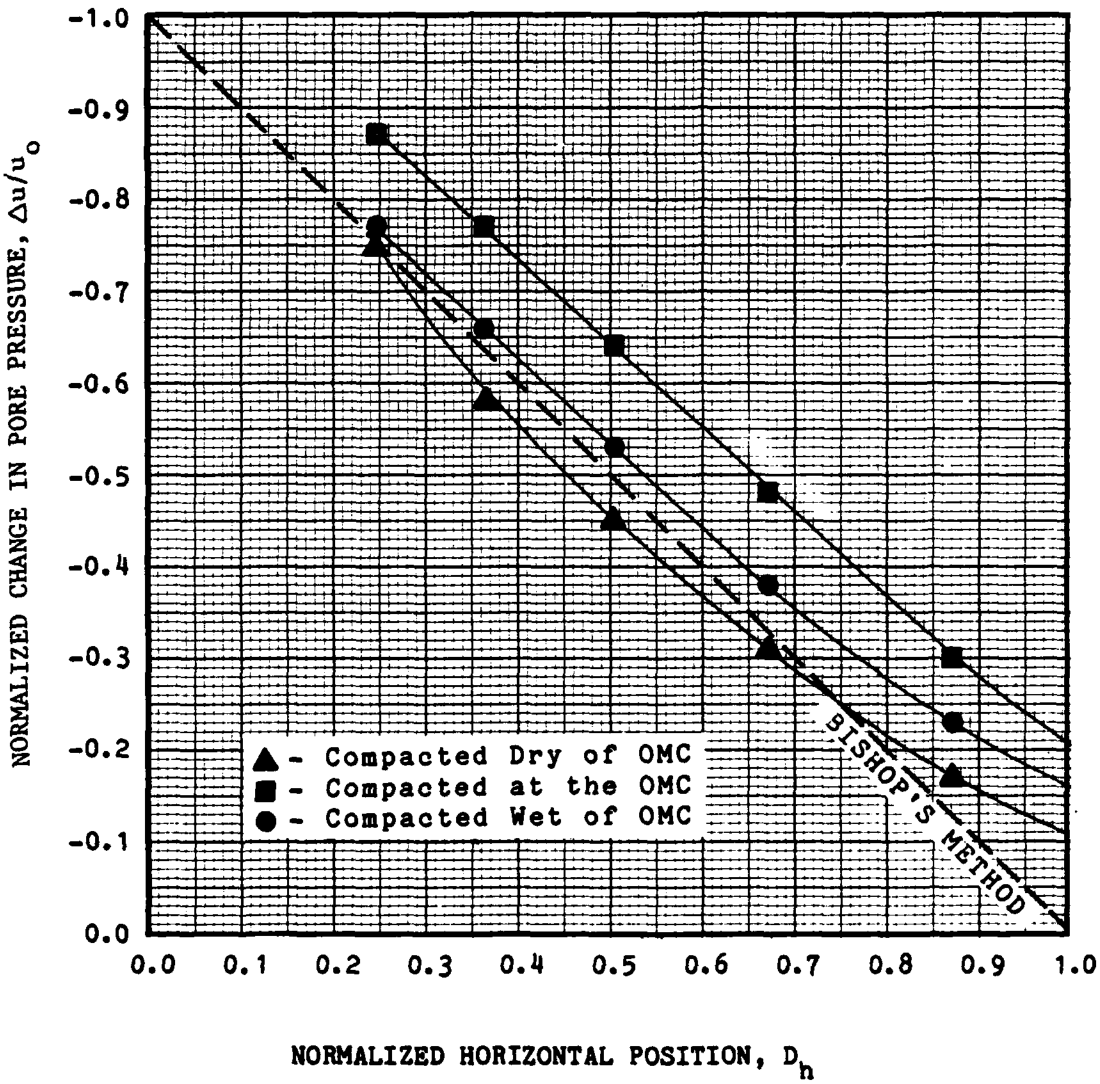


Fig. 6.5 Comparison Showing the Influence of Compaction Moisture Content on the Variation in the Change in Pore Water Pressure Calculated Using Measured Values of the Pore Pressure Coefficient for Saint Croix Clay Along a Representative Horizontal Plane at y/h=.20 in a 2:1 Slope 100 Feet High.

the soil is compacted at the optimum moisture content under Standard Proctor compaction criteria. To examine the influence of compactive effort at a given moisture content, changes in pore water pressure were computed using A values measured for specimens of Saint Croix clay compacted wet of the optimum moisture content under both Standard and Modified Proctor compaction criteria. A representative horizontal plane at y/h = .20 and a slope height of 100 feet was chosen for these comparisons. The changes in pore pressure are compared with those calculated using Bishop's method in Figure 6.6. The difference between the changes in pore water pressure computed using Bishop's procedure and those calculated using finite element stresses is greatest for the case of Modified Proctor compaction criteria.

Influence of slope height. Changes in pore water pressure were calculated using measured A values reported by Casagrande and Hirschfeld (1960) for Canyon Dam Clay assuming slope heights of 250 and 400 feet. These values are compared with changes in pore pressure calculated using Bishop's method in Figure 6.7 along a representative horizontal plane at y/h=.20. The trend seen in this figure is that, for a given soil, as slope height increases, the changes in pore pressure computed using finite element stresses become smaller, while those calculated using Bishop's method remain the same irrespective of the slope height.

Influence of upstream slope inclination. Changes in pore water pressure were calculated using measured A values reported by Lowe and Karafiath (1960) for Binga Dam Clay using upstream slope

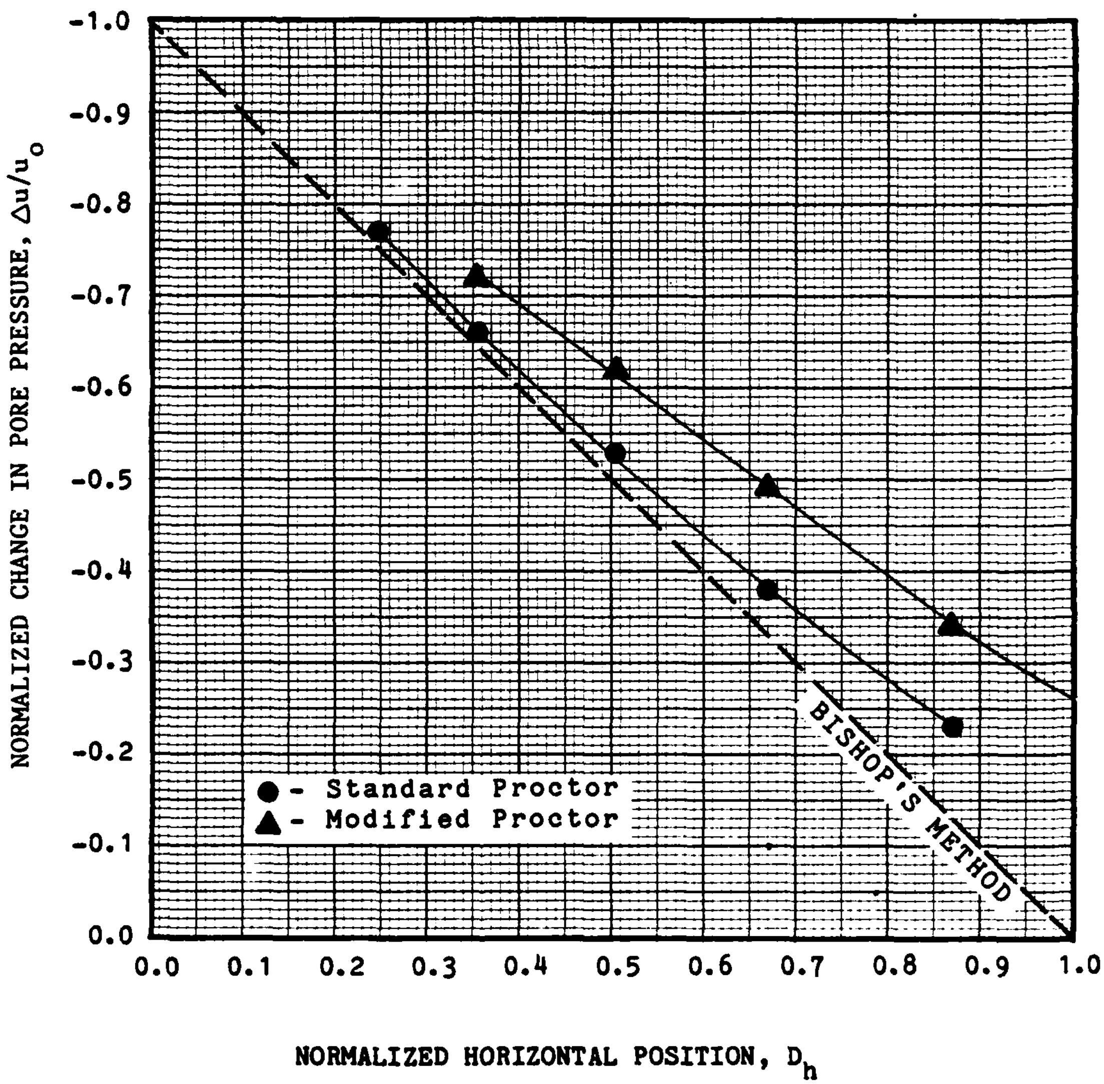


Fig. 6.6 Comparison Showing the Influence of Compactive Effort on the Variation in the Change in Pore Water Pressure Calculated Using Measured Values of the Pore Pressure Coefficient for Saint Croix Clay Along a Representative Horizontal Plane at y/h=.20 in a 2:1 Slope 100 Feet High.

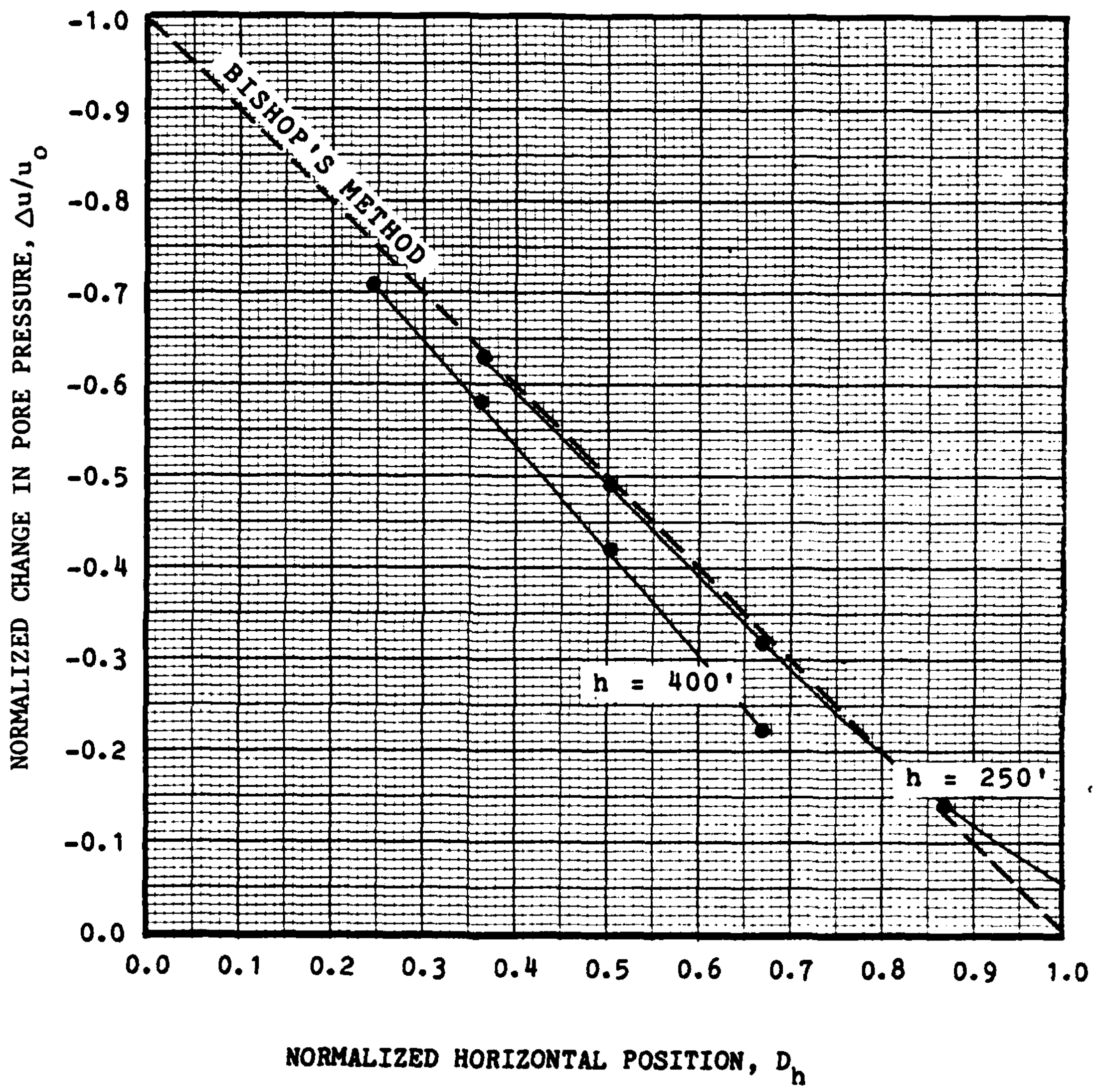


Fig. 6.7 Comparison Showing the Influence of Slope Height on the Variation in the Change in Pore Water Pressure Calculated Using Measured Values of the Pore Pressure Coefficient for Canyon Dam Clay Along a Representative Horizontal Plane at y/h = .20 in a 2:1 Slope.

inclinations of 2:1 and 4:1. These values are compared with changes in pore pressure calculated using Bishop's method in Figure 6.8 along a representative horizontal plane at y/h=.20. The trend seen in this figure is that, for a given soil, as the slope becomes flatter, the changes in pore pressure computed using Bishop's procedure remain the same while the changes in pore pressure computed using finite element stresses become larger.

Influence of the Intermediate Principal Change in Stress. In order to provide direct comparisons with Bishop's method, the preceding evaluations were conducted assuming axially symmetric loading using Skempton's A coefficient. To study the influence which the intermediate principal change in stress might have, changes in pore water pressure were computed using major, intermediate, and minor principal changes in stress from the finite element method. Measured values of the pore pressure coefficient based on octahedral stresses, a, for Binga Dam Clay were used in these computations assuming a dam height of 150 feet. Calculated values of the change in pore pressure are compared with values obtained assuming axially symmetric loading and using Bishop's method in Figure 6.9. Lower values of the change in pore pressure are obtained when the intermediate principal change in stress is used. However, it was shown previously in this report that there is no consistent trends which indicate that triaxial compression and triaxial extension define upper and lower bounds on all possible values of the pore pressure coefficient. The values of the pore pressure coefficient used to compute the changes in pore pressure

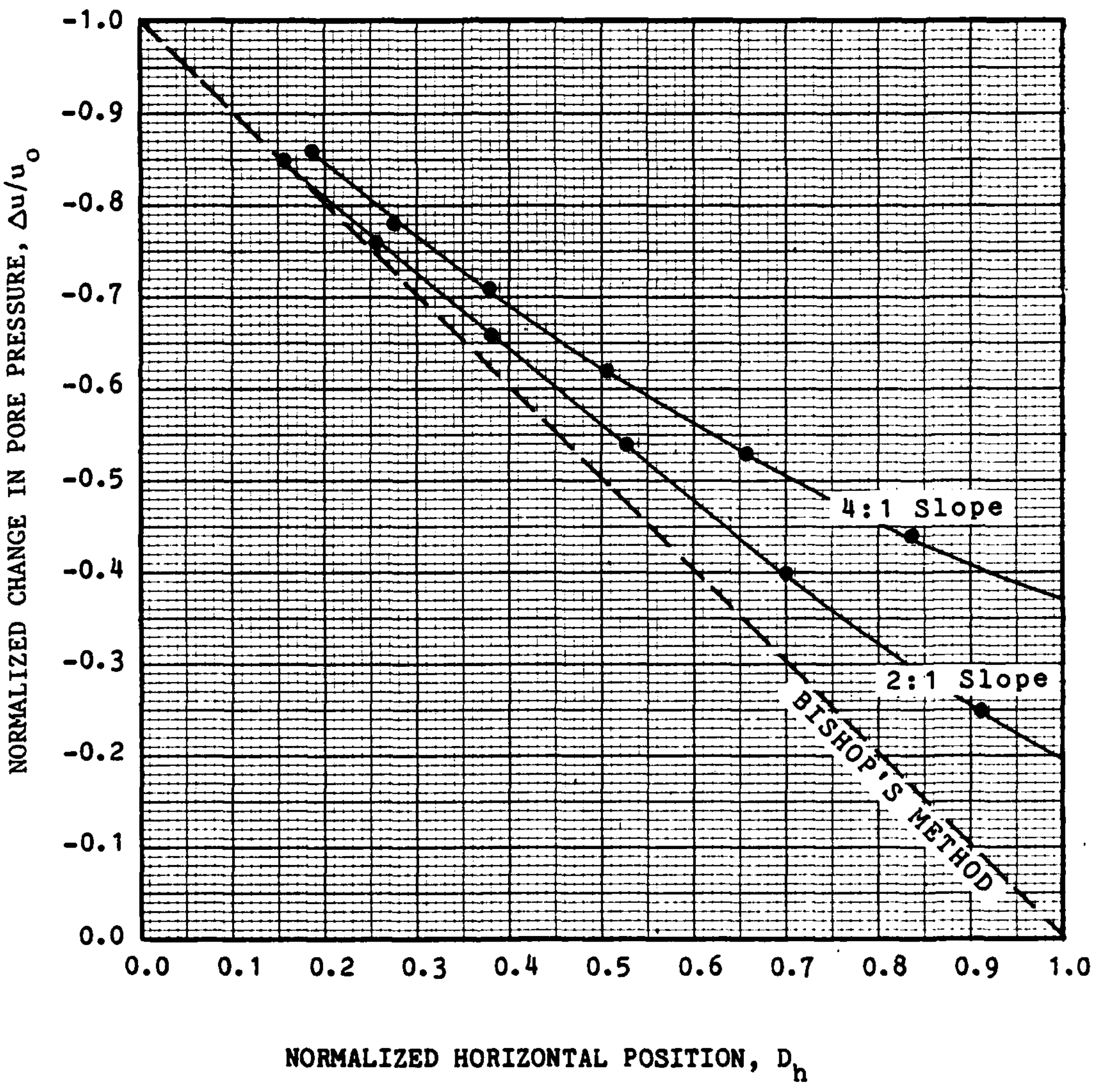


Fig. 6.8 Comparison Showing the Influence of Upstream Slope Inclination on the Variation in the Change in Pore Water Pressure Calculated Using Measured Values of the Pore Pressure Coefficient for Binga Dam Clay Along a Representative Horizontal Plane in a Slope 150 Feet High.

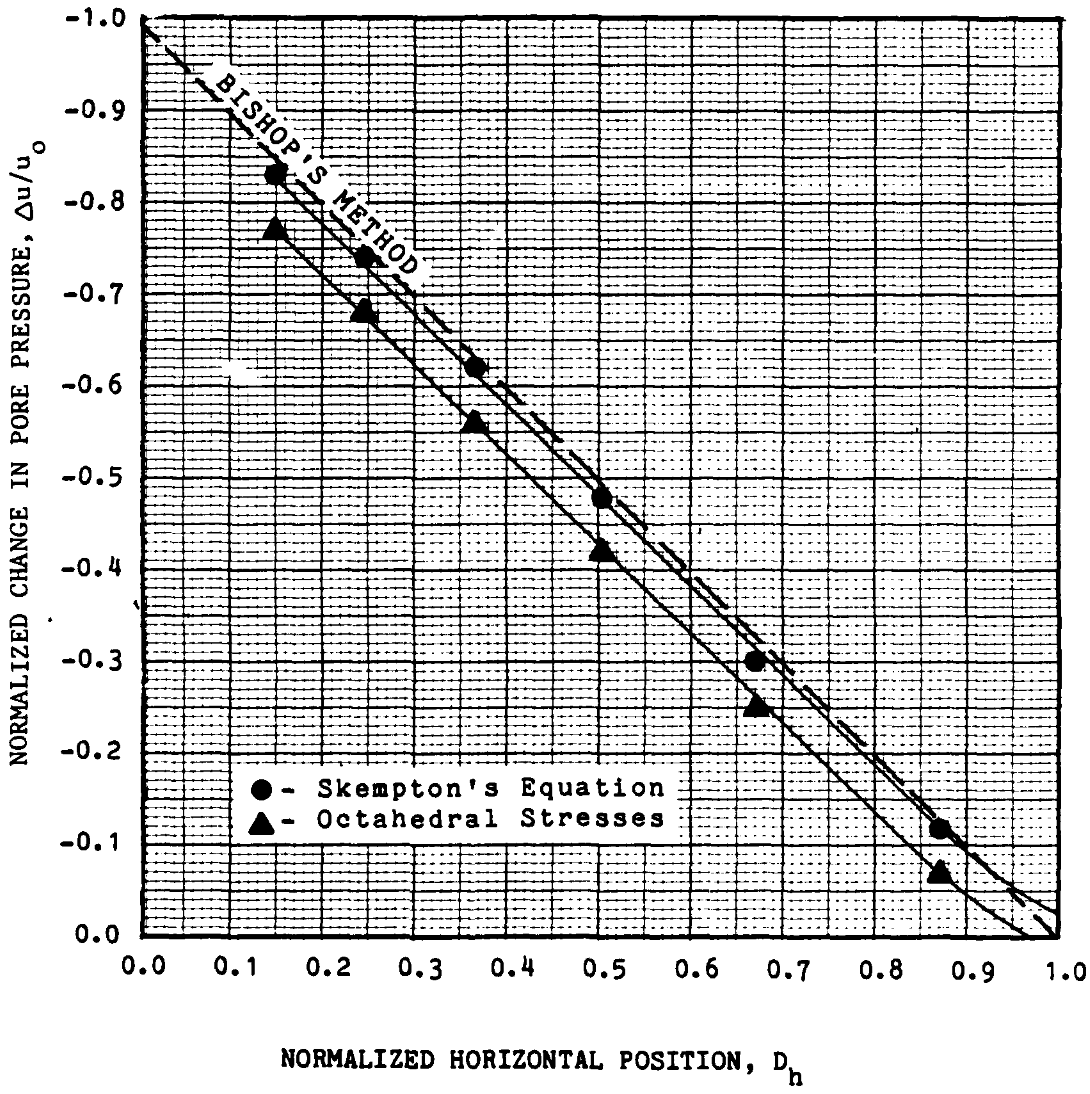


Fig. 6.9 Comparison Showing the Influence of the Intermediate Principal Change in Stress on the Variation in the Change in Pore Water Pressure Calculated Using Measured Values of the Pore Pressure Coefficient for Binga Dam Clay Along a Representative Horizontal Plane at y/h=.20 in a 2:1 Slope 150 Feet High.

in the comparisons presented in Figure 6.9 were obtained from triaxial compression tests. Since this study has shown that the value of the pore pressure coefficient could be higher or lower when other loading paths are used (Chapter 4), a firm conclusion concerning the accuracy of Bishop's method for plane strain loading cannot be reached. However, it appears that it is generally conservative to compute changes in pore pressure using major, intermediate and minor principal changes in stress from the finite element method and the pore pressure equation based on octahedral stresses.

Comparison of Measured and Required Values of the Pore Pressure Coefficient.

The comparisons presented in the previous sections were developed through an extensive, time-consuming series of computations involving stresses from the finite element method, normalized depth within the slope, and Skempton's pore pressure equation. A direct, qualitative analysis which permits an evaluation of the accuracy of Bishop's procedure can be obtained by comparing measured values of the pore pressure coefficient, A, with values of the pore pressure coefficient which would be required to give the changes in pore pressure computed using Bishop's procedure. This method is illustrated in Figure 6.10 in which measured values of the pore pressure coefficient for Binga Dam Clay (Lowe and Karafiath (1960)) have been superimposed on a plot relating required values of the pore pressure coefficient to normalized depth within the slope. Normalized depth for a given

measured pore pressure coefficient is calculated by dividing the confining pressure at which the soil was tested by the product of the soil's buoyant unit weight, γ', and the slope height, h. For the comparisons presented in Figure 6.10, the buoyant unit weight was assumed to be 70 pounds per cubic foot and the slope height used was 150 feet. In areas of the figure where measured values exceed the required values of the pore pressure coefficient, changes in pore pressure will be less than those calculated using Bishop's method. In areas of the plot where measured values are less than the "required" values, the changes in pore pressure will be greater than those calculated using Bishop's method. An analysis of the data presented if Figure 6.10 shows that, in a slope 150 feet high constructed with Binga Dam Clay, the changes in pore pressure in the lower 20 percent of the slope calculated using finite element stresses will be less than that computed using Bishop's method. A review of Figure 6.1 confirms this analysis.

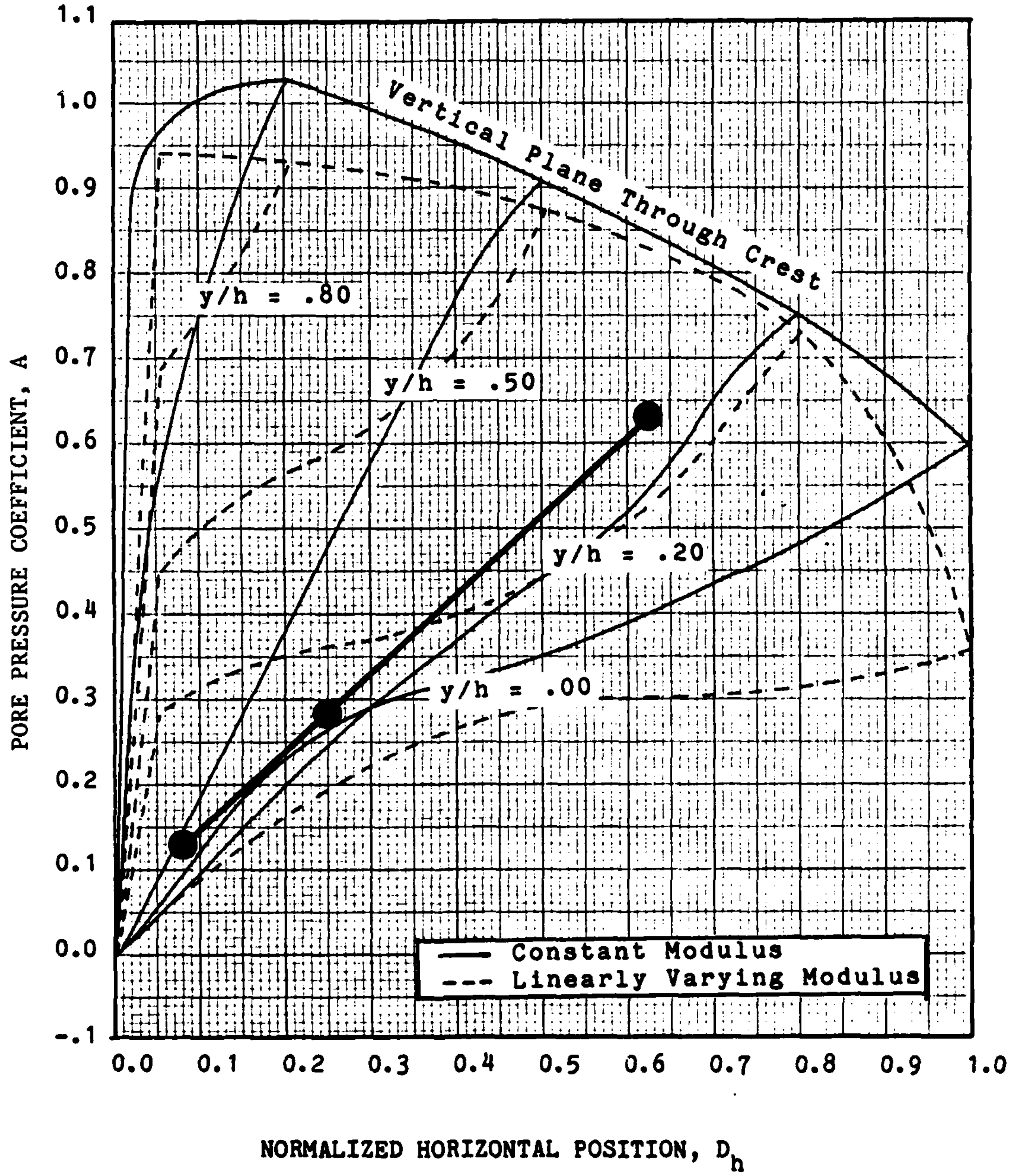


Fig. 6.10 Comparison of Measured Values of the Pore Pressure Coefficient for Binga Dam Clay with the Required Values of the Pore pressure Coefficient for an Assumed Slope Height of 150 Feet.

CONCLUSIONS

The changes in pore water pressure computed using Bishop's procedure were compared with changes in pore pressure calculated using measured values of the pore pressure coefficient and stresses from the finite element method. As intended by Bishop, it was found that, through the majority of a slope's cross-section, his method tends to provide lower and, hence, more conservative estimates of the changes in pore pressure induced in an earth slope as the result of rapid drawdown. However, this investigation has shown that, at some positions within the slope, changes in pore pressure calculated using Bishop's procedure are higher, and hence, less conservative, than those calculated using finite element stresses. Along representative horizontal planes within a slope, the variation in normalized values of the changes in pore pressure computed using Bishop's method remains constant, while normalized values of the changes in pore pressure calculated using finite element stresses increase at positions in the slope away from the base and away from the upstream face of the slope. At specific points within a slope, normalized values of the change in pore pressure computed using Bishop's method remain constant irrespective of slope height and upstream slope inclination, while normalized changes in pore pressure computed using finite element stresses decrease with increasing slope height and increasing slope inclination. The influence which compaction moisture content and compactive effort have on values of the pore pressure coefficient are

not accounted for in Bishop's method. However, the changes in pore pressure computed using finite element stresses are influenced by compaction moisture content and compactive effort. It is recommended that measured values of the pore pressure coefficient be compared with values of the pore pressure coefficient which would be required to compute the changes in pore pressure obtained using Bishop's method. This comparison will provide a qualitative estimate of areas within the slope where the changes in pore pressure computed using Bishop's will be greater than or less than the changes in pore pressure calculated using finite element stresses and measured values of the pore pressure coefficient. While Bishop's method will provide a quick, usually conservative estimate of the changes in pore water pressure induced in an earth slope by rapid drawdown, it is apparent that, to achieve accurate estimates, a more rationale approach must be taken.

CHAPTER 7

SUMMARY AND CONCLUSIONS

Statement of the Problem.

The application of effective stress methods to geotechnical design problems involving undrained loading is limited by our ability to accurately predict the pore water pressure changes which occur as the result of a change in stress. A number of researchers (Skempton (1954 and 1960), Bishop (1954), Henkel (1960), and Henkel and Wade (1966)) have proposed that, during undrained shear, changes in pore water pressure can be related to principal changes in stress through empirically derived pore pressure coefficients. The main objective of this study was to evaluate the use of pore pressure coefficients in predicting the pore water pressure changes which are induced in earth slopes by the rapid lowering of the water level adjacent to the slope.

Pore Pressure Coefficient Equations

Two different forms of the pore pressure coefficient equation and, hence, the pore pressure coefficient have been proposed in the literature. Skempton (1954) expressed the change in pore water pressure in terms of the major and minor principal changes in stress and the pore pressure coefficient, A. Subsequently, Henkel (1960) and Skempton (1960) proposed expressing the change in pore water pressure

in terms of the major, intermediate, and minor principal changes in stress and the pore pressure coefficient, a. For the special case of axially symmetric loading, a numerical equivalence exists between these two coefficients and it is convenient to express a in the form $a\sqrt{2}$.

Pore Pressure Coefficients.

One of the objectives of this study was to determine if either of the pore pressure coefficients, A or a, were uniquely independent of loading path or to determine if either one was less dependent on loading path than the other. A literature search was conducted to collect appropriate undrained shear test data. The majority of the data were from triaxial compression and triaxial extension tests. Based on the available data, it was shown that a unique relationship between the pore pressure coefficients which is independent of load path does not exist. On the average, values of $a_f\sqrt{2}$ for triaxial compression are .14 higher than values for triaxial extension. Values of A_f are, on the average, .19 lower for triaxial compression compared with values for triaxial extension. The differences found in the two forms of the pore pressure coefficient for triaxial compression and triaxial extension did not appear to be related to strain at failure, initial effective principal stress ratio, overconsolidation ratio, confining pressure, or type of specimen. An evaluation of the influences of plane strain on values of the pore pressure coefficient, A_f, indicated that triaxial compression and triaxial extension do not

establish upper and lower bounds on possible values of the pore pressure coefficient. In addition, available data indicate that variations in values of the pore pressure coefficient do not appear to be related to the magnitude of the intermediate principal change in stress.

Finite Element Computations

Linear elastic finite element computations were performed to estimate the changes in stress induced in an earth slope by rapid drawdown. Several sets of computations were performed to evaluate the effect of soil modulus and slope inclination on the changes in stress. The assumed variation in soil modulus had a significant influence on the major principal change in stress and a negligible influence on the minor principal change in stress. The assumption of a modulus which was constant throughout the slope gave computed values of the major principal change in stress which were considerably smaller than those computed assuming that the modulus increased linearly with depth. It was also found that the principal stress difference is effected by upstream slope inclination. As the upstream slope becomes flatter, values of the principal stress difference will be smaller.

Comparisons of Changes in Pore Pressure Computed Using Finite Element Stresses and Changes in Pore Pressure Calculated Using Bishop's Method.

An investigation was conducted to evaluate the assumptions

which Bishop (1954) made in developing his procedure to estimate the changes in pore water pressure induced in an earth slope by rapid drawdown. Values of the major principal change in stress assumed by Bishop are higher than those calculated through linear elastic finite element analysis. In addition, Bishop's assumption that the pore pressure coefficient can be assigned a constant value of one throughout the slope was shown to be unrealistic. The evaluations conducted in this study based on stresses from the finite element method show that the values of the pore pressure coefficient would have to vary throughout the slope to produce the changes in pore water pressure calculated using Bishop's procedure.

Pore water pressure changes were calculated using measured values of the pore pressure coefficient and stresses computed from finite element analyses. These values were compared with the changes in pore pressure calculated using Bishop's method. Generally, it was found that Bishop's method tends to produce smaller estimates and, therefore, to provide conservative estimates of the changes in pore pressure induced by rapid drawdown. However, at some points within a slope, changes in pore pressure computed using Bishop's procedure are higher than the changes in pore pressure calculated using finite element stresses. Several trends were identified in the comparisons which were conducted between normalized values of the changes in pore pressure computed using Bishop's method and normalized values of the changes in pore pressure calculated using finite element stresses. Irrespective of the height above the base of the slope, the variation

in the normalized values of the changes in pore pressure computed using Bishop's method is identical for any horizontal plane considered. However, normalized values of the change in pore pressure calculated using finite element stresses become increasingly higher at points away from both the base and the face of the slope. Normalized values of the change in pore pressure calculated using Bishop's method are not influenced by either slope height or upstream slope inclination. At points within the slope, normalized values of the change in pore pressure computed using finite element stresses decrease with both increasing slope height and increasing upstream slope inclination. The influence which soil moisture content and compactive effort may have on values of the change in pore pressure is not considered in Bishop's method. It was shown that both compactive effort and soil moisture content influence both the magnitude and the variation in the changes in pore pressure calculated using finite element stresses and measured values of the pore pressure coefficient. It is recommended that measured values of the pore pressure coefficient be compared with values of the pore pressure coefficient which would be required to compute the changes in pore pressure obtained from Bishop's method. This comparison will provide a qualitative estimate of areas in the slope where changes in pore pressure calculated using Bishop's method will be greater than and less than changes in pore pressure calculated using finite element stresses and measured values of the pore pressure coefficient. Bishop's method should only be used to provide an initial approximation of rapid drawdown induced pore pressures.

RECOMMENDATIONS FOR FUTURE RESEARCH

Methods currently available which use the pore pressure equation to estimate the changes in pore pressure induced in an earth slope as the result of rapid drawdown have been developed using simple, conservative assumptions regarding the values of both the principal changes in stress and the value of the pore pressure coefficient used in the pore pressure equation. This study has shown that a rational approach to using the pore pressure equation in rapid drawdown analyses may be achieved by using principal changes in stress calculated using the finite element method and measured values of the pore pressure coefficient. In order to verify this possibility, measured values of the principal changes in stress and the changes in pore pressure which occur in an earth slope during an actual rapid drawdown are required. In addition, it is recommended that slope stability analysis be conducted to compare the factor of safety obtained using Bishop's method and the factor of safety obtained using the pore pressure equation with finite element stresses and measured pore pressure coefficients. This study has shown that additional research into the fundamental mechanisms of pore pressure development in soils subjected to undrained shear is required. Nonlinear finite element analysis using an elasto-plastic stress-strain model hold the potential to predict changes in pore pressure due to rapid drawdown directly, without the need for empirical pore pressure coefficients.

APPENDIX A

COMPUTER CODE VERIFICATION, MESH ANALYSIS AND MODULUS SENSITIVITY ANALYSIS

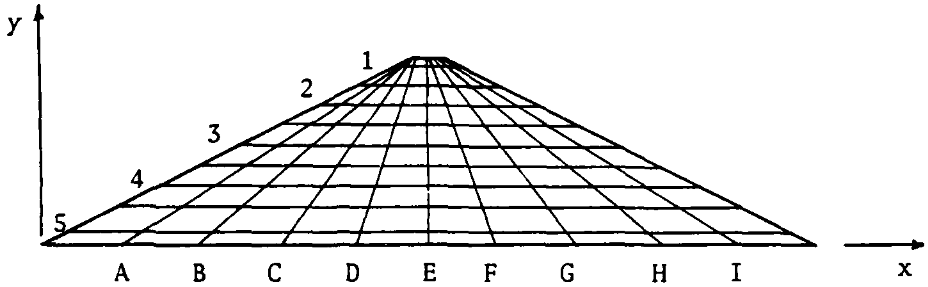
The finite element analysis which were conducted as a part of this study employed the computer code, TEXGAP, which was developed by Becker and Dunham (1979). TEXGAP is a finite element program designed for the analysis of two-dimensional, linearly elastic plane or axisymmetric bodies. The code includes an extensive element library. An eight-noded isoparametric quadrilateral element designated "QQH" by Becker and Dunham was selected for use in this study. This element is a conventional displacement element which was reformulated to include the mean pressure along the side of the element as an independent variable. The order of interpolation is quadratic for displacements and linear for the mean pressure, giving a total of twenty degrees of freedom for this element. Sixteen degrees of freedom are used to specify displacements at the eight nodes and the remaining four degrees of freedom are used to specify the mean pressure on each of the four sides of the element. The code also provides an extremely flexible system of grid refinement options. Because stress changes in the upstream portion of earth slopes was of primary concern for this study, a graduated mesh was selected which concentrated nodal points in that portion of the slope. A description of evaluations conducted

to verify code input, to establish an appropriate element mesh configuration, and to determine the sensitivity of the finite element computations to assumed variation in modulus follow.

CODE VERIFICATION

To insure that input data had been correctly formatted, several computer runs were accomplished in which an identical problem was analyzed using TEXGAP and PRONEW, a two-dimensional finite element program developed by Dr. Stephen G. Wright. PRONEW uses four noded quadrilateral elements and stresses are calculated and output at the integration points. In TEXGAP, strains and stresses are computed at the integration points and are output at the corner nodes using bilinear extrapolation within each element. Since more than one value of a stress was provided for adjacent elements in TEXGAP, all values provided were averaged giving a single value of stress at each corner node. To eliminate the need for data extrapolation, different finite element grids were chosen so that the corner nodal points in the TEXGAP grid would coincide with the integration points in the PRONEW grid. The TEXGAP grid employed 45 elements with 164 nodal points while the PRONEW grid had 99 elements and 120 nodal points. As a result, the TEXGAP grid provided a higher degree of accuracy than the PRONEW grid. The results of the verification runs are provided in Table A.1. To facilitate data comparison, a dam cross section showing grid coordinates is provided at the top of the table. A comparison of

Table A.1 - Summary of Finite Element Analysis Results Used for Code Verification Comparisons



a) Normal Stress in the x-Direction (σ_x) (psf)

PLANE	Y (FT)	A	B	C	D	TEXGAP PRONEW E	F	G	H	I
1	93.75	342	307	282	255	239	224	231	236	234
		293	268	250	228	210	196	185	171	159
2	68.75	1399	1191	1036	910	808	669	579	524	522
		1430	1196	1055	905	776	651	588	521	538
3	43.75	2186	1787	1442	1160	919	699	546	467	451
		2197	1780	1467	1157	921	684	571	458	457
4	18.75	3094	2384	1696	1138	724	447	315	283	271
		3057	2366	1713	1092	761	416	338	278	267
5	6.25	3759	2800	1851	1016	465	242	216	211	164
		3709	2754	1839	950	576	192	203	228	175

B) Normal Stress in the y-Direction (σ_y) (psf)

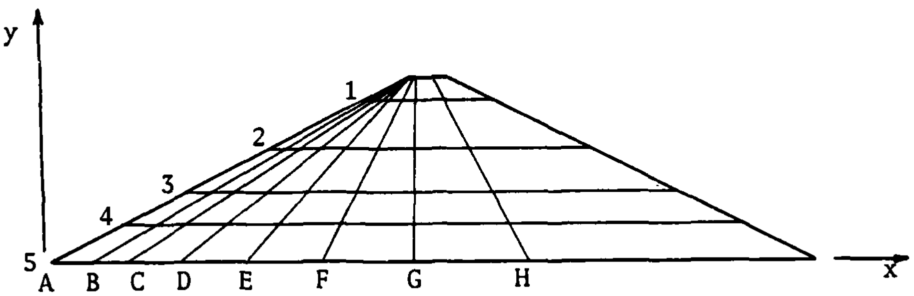
PLANE	Y (FT)	A	B	C	D	TEXGAP PRONEW E	F	G	H	I
1	93.75	323	160	-1	-84	-15	-98	-42	35	79
		214	66	19	-16	-7	-8	-12	-13	11
2	68.75	1402	951	556	219	26	-24	2	33	81
		1404	914	520	141	39	-64	-24	25	90
3	43.75	2571	1797	1069	478	120	-3	22	88	132
		2555	1796	1086	381	150	-64	23	114	131
4	18.75	3700	2676	1650	773	233	62	110	171	141
		3668	2650	1673	660	300	-8	111	201	133
5	6.25	4221	3084	1955	938	303	119	169	192	124
		4215	3030	1962	830	398	44	157	220	124

normal stresses in the x-direction is contained in Table A.1.a while those in the y-direction is provided in Table A.1.b. At each grid point, the value of the stress estimated using TEXGAP is printed directly above the stress estimated using PRONEW. This comparison shows good agreement in the stresses estimated by the two programs. The differences in stress values is due primarily to the differences in the degrees of freedom associated with the two grids used. The results show that data input for TEXGAP is correct.

MESH ANALYSIS

Mesh analysis was conducted using TEXGAP, to develop a mesh geometry which would minimize computer execution time without reducing the accuracy of the analysis. In this analysis, stresses computed using a coarse mesh (81 elements and 280 nodes) were compared with stresses computed using a fine mesh (324 elements and 1045 nodes). The execution times for the fine mesh was 283 seconds versus 57 seconds for the coarse mesh. The results of that comparison are provided in Table A.2 for a 2:1 upstream slope. Values of normal stress in the x-direction are compared in Table A.2.a and values of the normal stress in the y-direction are compared in Table A.2.b. Values calculated using the fine mesh are shown directly above those from the coarse mesh. The data shown was obtained from coincident nodal points in the two grids. As can be seen, little change is evident in the stresses for the two grids. To insure that upstream slope does not

Table A.2 - Summary of Finite Element Analysis Results Used for Optimum Mesh Size Determination Assuming a 2:1 Upstream Slope



a) Normal Stress in the x-Direction (σ_x) (psf)

FINE MESH
COARSE MESH

PLANE	Y (FT)	A	B	C	D	E	F	G	H
1	89.50	583	545	507	471	438	397	348	303
		584	547	508	473	440	399	349	304
2	64.38	1851	1710	1552	1383	1212	1044	860	668
		1860	1713	1555	1385	1214	1047	864	665
3	38.05	2821	2611	2346	2026	1674	1318	966	626
		2837	2600	2338	2020	1671	1316	972	616
4	20.77	3629	3334	2963	2493	1942	1362	833	442
		3493	3305	2944	2480	1925	1356	839	426
5	0.00	5464	4713	4018	3266	2393	1358	426	141
		5389	4625	3988	3248	2385	1344	379	127

B) Normal Stress in the y-Direction (σ_y) (psf)

FINE MESH
COARSE MESH

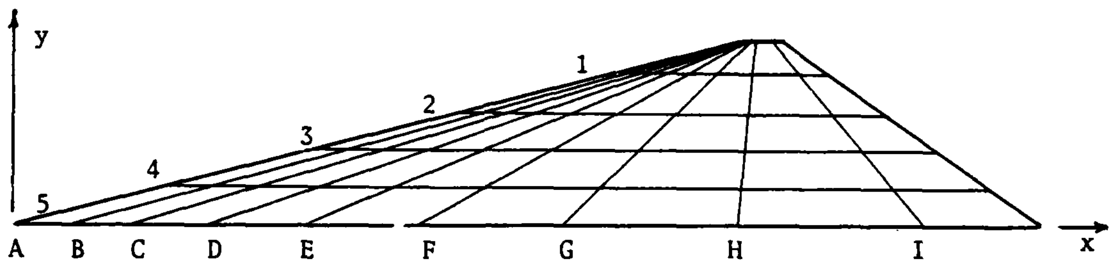
PLANE	Y (FT)	A	B	C	D	E	F	G	H
1	89.50	637	530	406	267	130	33	-8	-21
		634	530	406	264	124	27	-11	-23
2	64.38	2129	1885	1584	1219	799	369	55	-47
		2131	1886	1584	1219	794	357	38	-48
3	38.05	3604	3229	2779	2224	1543	789	178	-8
		3608	3227	2780	2227	1540	772	142	-27
4	20.77	4614	4093	3521	2854	2038	1087	280	51
		4584	4095	3525	2857	2038	1069	227	34
5	0.00	6021	5143	4367	3545	2600	1471	453	153
		5984	5111	4364	3549	2604	1448	391	132

affect the agreement in stresses obtained for a 2:1 slope, a similar comparison was conducted for a 4:1 slope. The results of that comparison are provided in Table A.3. Again, it can be seen that the agreement in estimated stresses is excellent. As a result of this analysis, a refinement of the coarse mesh was selected for the finite element analysis conducted in this study.

MODULUS SENSITIVITY ANALYSIS

In selecting values of soil modulus to be used in this study, a primary goal was to select values which represent upper and lower limits on all possible values which could be assumed in a linear, elastic analysis. Initially, it was estimated that assumptions of a constant modulus throughout the slope and a modulus that varied linearly with depth would achieve that goal. In order to verify that assumption, five distinct finite element analysis of the 2:1 slope configuration were conducted in which only the assumed value of modulus was varied. The first step was to check the influence of magnitude using a constant value of soil modulus. Finite element computations were conducted in which constant soil modulus values of 100,000 psf and 1,000,000 psf were used. The values of change in stress at all nodes within the finite element mesh were identical. A similar check was conducted assuming that the modulus varied linearly with depth. In this case, verification runs were conducted in which the soil modulus was varied from 0.0 to 200,000 and 0.0 to 2,000,000 psf.

Table A.3 - Summary of Finite Element Analysis Results Used in Optimum Mesh Size Determination Assuming a 4:1 Upstream Slope



a) Normal Stress in the x-Direction (σ_x) (psf)

FINE MESH
COARSE MESH

PLANE	Y (FT)	A	B	C	D	E	F	G	H	I
1	89.50	1060	994	915	822	714	598	490	383	297
		1065	999	921	827	719	603	492	384	285
2	64.38	1806	1695	1561	1399	1207	989	766	565	382
		1802	1691	1557	13978	1206	988	765	566	361
3	38.05	3135	2901	2626	2303	1923	1484	1018	620	314
		3126	2893	2618	2296	1916	1480	1013	616	296
4	20.77	4221	3888	3497	3038	2502	1879	1175	549	217
		4206	3876	3486	3029	2494	1870	1161	524	206
5	0.00	5728	5260	4711	4065	3312	2443	1432	333	127
		5722	5251	4700	4054	3303	2434	1434	262	105

B) Normal Stress in the y-Direction (σ_y) (psf)

FINE MESH
COARSE MESH

PLANE	Y (FT)	A	B	C	D	E	F	G	H	I
1	89.50	1083	986	869	726	553	347	120	-11	-12
		1080	986	869	726	554	344	111	-25	-11
2	64.38	2197	2020	1808	1552	1242	863	417	36	-1
		2196	2020	1807	1553	1243	863	406	6	0
3	38.05	3821	3515	3152	2725	2221	1616	874	143	44
		3819	3514	3152	2725	2221	1619	867	89	42
4	20.77	4899	4505	4038	3489	2846	2095	1187	228	89
		4897	4504	4037	3488	2845	2096	1184	163	77
5	0.00	6209	5703	5107	4407	3591	2649	1555	359	136
		6212	5702	5106	4405	3588	2649	1549	292	110

Again, the values of change in stress were identical at all nodal points. As anticipated, these analyses verified that the changes in stress estimated based on Hooke's law are not sensitive to the absolute magnitude of the modulus value used in the computation. However, estimated changes in stress were found to be sensitive to the range in values used for a linearly varying modulus. To verify that assumptions of a constant modulus and a linearly varying modulus with an initial value of zero define the boundaries of all variations in modulus which could be assumed, a finite element analyses was performed in which the modulus was varied linearly from 200,000 psf to 2,000,000 psf. The influence of assumed variation in soil modulus on change in stress along a horizontal plane through the upstream portion of a 2:1 slope is presented in Figure A.1 and A2. The influence on the major principal change in stress is shown in Figure A.1 while that for the minor principal change in stress is shown in Figure A.2. In both figures, the change in stress estimated by assuming that the modulus varies from 200,000 to 2,000,000 psf is bounded by the values estimated assuming a constant modulus and a modulus increasing linearly from zero. This evaluation shows that the use of a constant modulus and a linearly varying modulus with an initial value of zero provided an upper and lower bound on the possible range in values of the principal changes in stress which were calculated in this study using the finite element method.

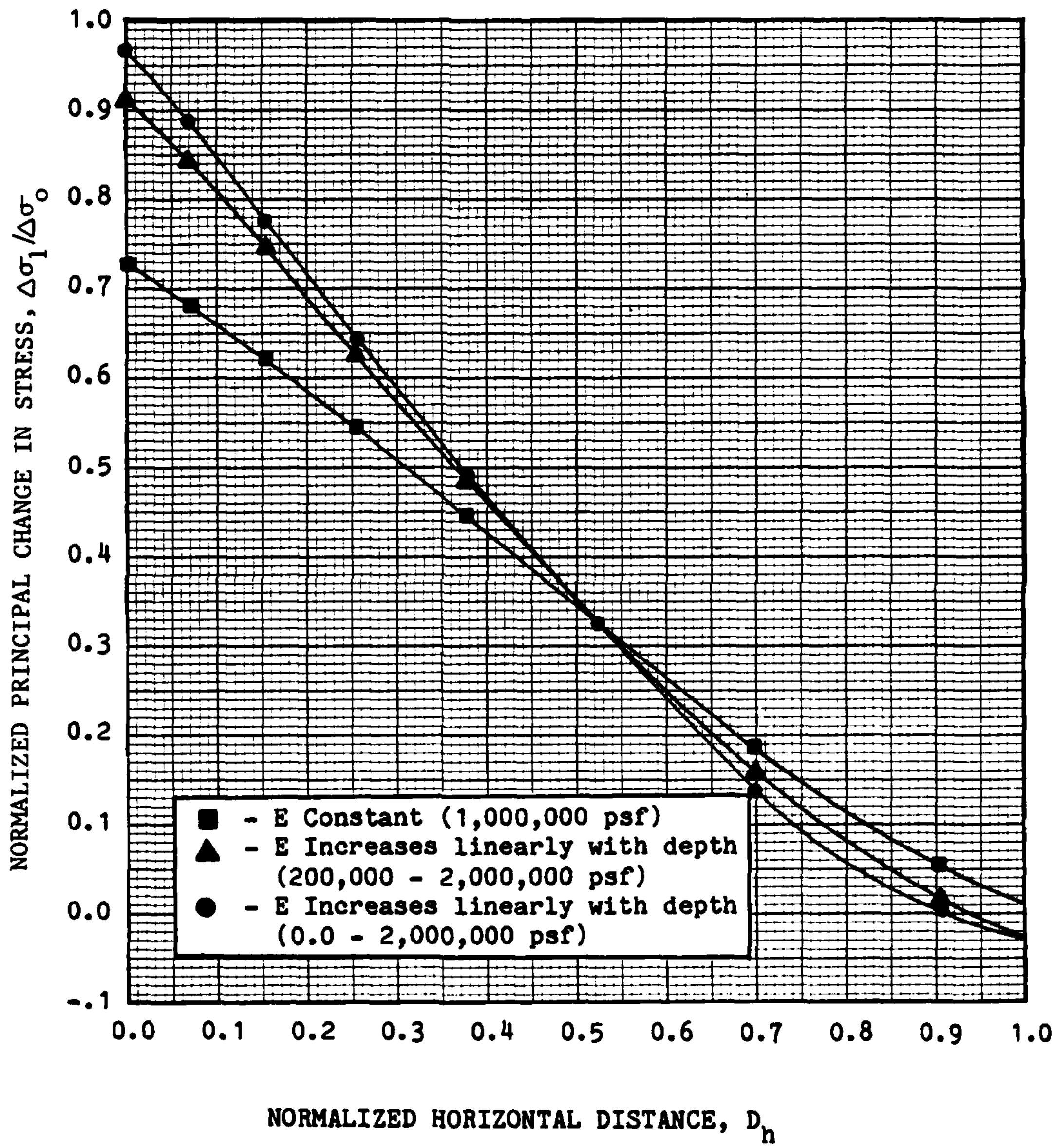


Fig. A1 Influence of Assumed Variation in Soil Modulus on the Major Principal Change in Stress.

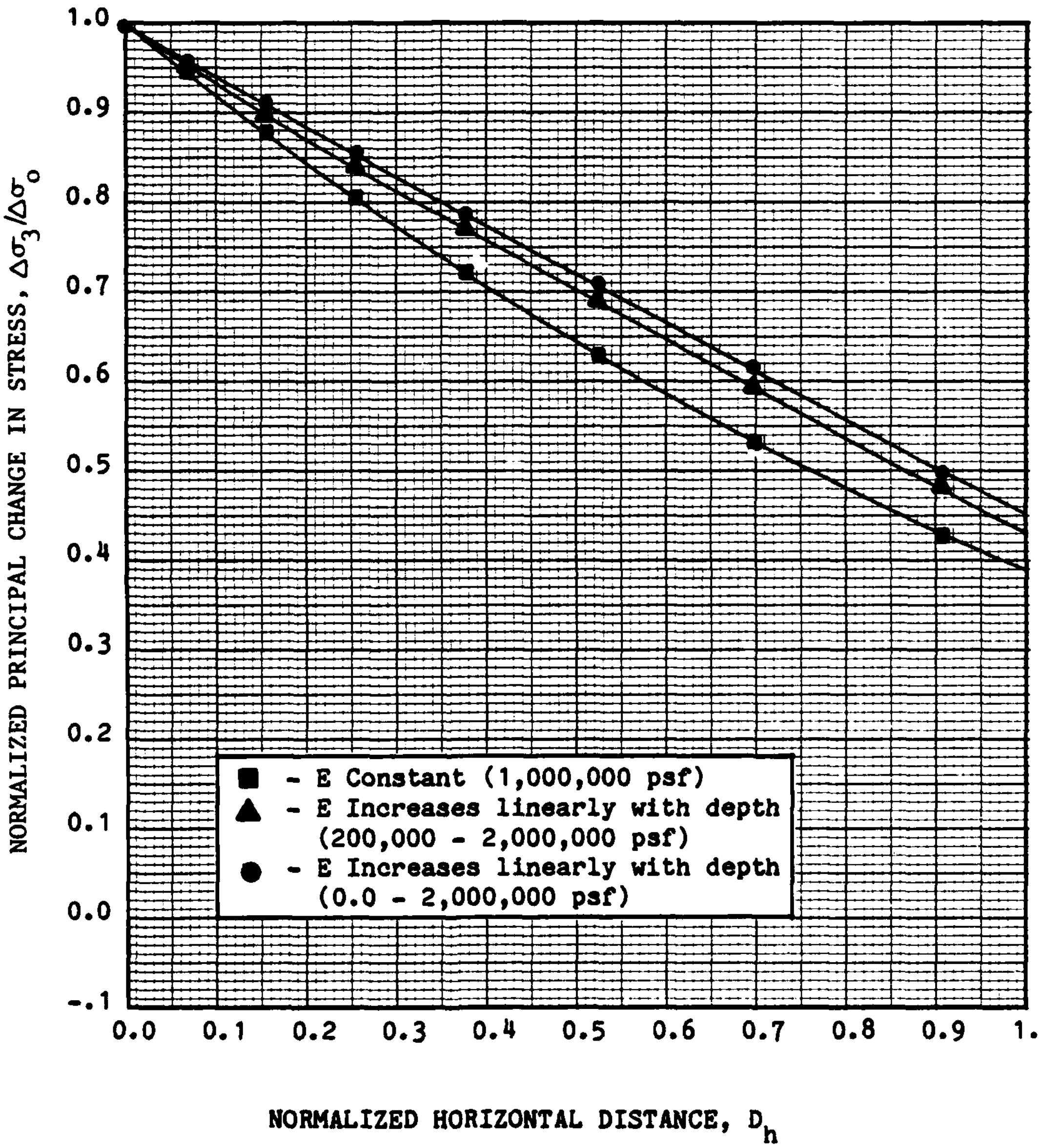


Fig. A2 Influence of Assumed Variation in Soil Modulus on the Minor Principal Change in Stress.

APPENDIX B

SHEAR TEST DATA

An effort was made to find all published reports of undrained soil shear testing for which pore pressure measurements were made and more than one loading path to failure had been used. To aid in this search, various indexes and information retrieval systems were examined, e.g.,Geodex, and indexes for ASCE , Geotechnique, and the proceedings of the International Soil Mechanics and Foundation Engineering Conferences. In addition, papers containing summaries of shear test results were reviewed and their reference lists were used to trace down source papers.

The data selected for use in this study are summarized in the data sheets which follow. Each data sheet contains shear test results obtained from one source. All test data reported by the author were extracted from the original publication and summarized on these sheets in the section below SOIL TESTED. This data was then used to calculate the information which was required for this study and is shown in the data sheet section entitled PORE PRESSURE COEFFICIENT PARAMETERS. The details of all interpretations made in reducing the data from the original source is presented in the remarks section of the summary sheet.

SHEAR TEST DATA SUMMARY

REFERENCE: Saxena, Hedberg and Ladd (1974) **PAGE:** 1/1

SOIL TESTED: Hackensack Valley Varved Clay

INDEX PROPERTIES

w_n: 49% PI : ______ ϕ_C: 18

w_p: ______ σ'_{vo}: 1.28 ϕ_E: 19

w_l: 36% σ'_{vm}: 2.50 G_s: ______

SPECIMEN PREPARATION/SAMPLING: Undisturbed/Osterberg Piston Samples

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial **Units Used by Author:** kg/cm^2

TEST NO.	K	OCR	ϵ_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CK_oUC	.65	1.0	.6	3.00	.345	0	0	.466
CK_oUE	.65	1.0	13.1	3.00	0	0	-1.92	.126
CIUC1	1.00	1.0	3.2	3.00	1.20	0	0	1.93
CIUC2	1.00	1.0	4.0	3.00	1.32	0	0	1.87

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CKoUC	1	42.7	1.35	1.02
CkoUE	1	42.7	1.07	.40
CIUC1	1	42.7	1.61	1.28
CIUC2	1	42.7	1.42	1.09

REMARKS: 1. Strain controlled tests (.5%/hour).
2. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Norwegian Geotechnical Institute (1975) PAGE: 1/1

SOIL TESTED: Drammen Clay

INDEX PROPERTIES

w_n:	52%	PI :	27	ϕ_C:	
w_p:	28%	σ'_{vo}:	15-20	ϕ_E:	
w_l:	55%	σ'_{vm}:	40	G_s:	2.78

SPECIMEN PREPARATION/SAMPLING: Undisturbed/Piston Tube Samples

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial Units Used by Author: ton/m^2

TEST NO.	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
TB1(2)	1	4.0	6.0	10	21.0	0	0	1.7
TB1(3)	1	4.0	5.5	10	20.4	0	0	2.7
TB1(4)	1	4.0	5.6	10	21.4	0	0	2.5
TB1(5)	1	4.0	5.0	10	22.0	0	0	2.3
TB1(29)	1	4.0	15.0	10	0	0	-10.6	-3.1

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
TB1(2)	1	14.2	.08	-.25
TB1(3)	1	14.2	.13	-.20
TB1(4)	1	14.2	.12	-.22
TB1(5)	1	14.2	.10	-.23
TB1(29)	1	14.2	.71	.04

REMARKS: 1. Strain controlled tests (3%/hour).
2. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: K.T. Law and R.D. Holtz (1978) PAGE: 1/2

SOIL TESTED: Kars Clay

INDEX PROPERTIES

w_n: ______	PI : ______	ϕ_C: ______
w_p: ______	σ'_{vo}: ______	ϕ_E: ______
w_l: ______	σ'_{vm}: ______	G_s: ______

SPECIMEN PREPARATION/SAMPLING: Undisturbed/Piston Samples

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial **Units Used by Author:** kN/m^2

TEST NO.	K	OCR	e_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
AC	.75	1.0	1.1	73.4	82.8	0	0	28.1
LC	.75	1.0	-2.0	73.4	88.2	88.2	0	65.4
AE	.75	1.0	-1.6	73.4	0	0	-87.4	-23.8

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
AC	1	10.6	.34	.01
LC	1	10.6	.74	.07
AE	1	10.6	.73	.06

REMARKS: 1. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: K.T. Law and R.D. Holtz (1978) PAGE: 2/2

SOIL TESTED: Gloucester Clay

INDEX PROPERTIES

w_n:______ PI : ______ ϕ_C:______
w_p:______ σ'_{vo}: ______ ϕ_E:______
w_l:______ σ'_{vm}: ______ G_s:______

SPECIMEN PREPARATION/SAMPLING: Undisturbed/Piston Samples

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial Units Used by Author: kN/m^2

TEST NO.	K	OCR	ε_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
AC	.80	1.0	-	73.4	77.4	0	0	31.0
LC	.80	1.0	-	73.4	87.4	87.4	0	69.9
AE	.80	1.0	-	73.4	0	0	-88.4	-17.7

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
AC	1	10.6	.40	.07
LC	1	10.6	.80	.13
AE	1	10.6	.80	.13

REMARKS: 1. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Leon and Alberro (1977) PAGE: 1/1

SOIL TESTED: Mexico City Clay

INDEX PROPERTIES

w_n: 391%	PI : 271	ϕ_C: 44
w_p: 86%	σ_{vo}': 4.8	ϕ_E: 51
w_l: 357%	σ_{vm}':	G_s:

SPECIMEN PREPARATION/SAMPLING: Undisturbed block samples

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial Units Used by Author: kg/cm^2

TEST NO.	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
AC	1	1.0	7.8	0.50	0.56	0	0	0.38
AC	1	1.0	10.6	1.00	1.08	0	0	0.83
AC	1	1.0	5.2	2.00	1.75	0	0	1.66
AE	1	1.0	4.6	0.50	0	0	-0.59	-0.01
AE	1	1.0	5.1	1.00	0	-229	-0.95	-0.02
AE	1	1.0	5.1	2.00	0	-214	-1.96	-0.08
AE	1	1.0	4.0	4.00	0	-200	-3.82	-0.14

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
AC	1	7.1	.69	.35
AC	1	14.2	.77	.44
AC	1	28.3	.95	.61
AE	1	7.1	.98	.32
AE	1	14.2	.98	.31
AE	1	28.3	.96	.29
AE	1	35.4	.96	.30

REMARKS: 1. Strain controlled tests (.041%/hr).
2. B assumed equal to 1 (not reported by author).

SHEAR TEST DATA SUMMARY

REFERENCE: Vaid and Campanella (1974) PAGE: 1/1

SOIL TESTED: Haney Clay

INDEX PROPERTIES

w_n:	PI : 18%	ϕ_C: 21.4
w_p: 26%	σ'_{vo}:	ϕ_E: 33.8
w_l: 44%	σ'_{vm}:	G_s:

SPECIMEN PREPARATION/SAMPLING: Undisturbed, trimmed block samples

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial-Plane Strain Units Used by Author: kg/cm^2

TEST NO.	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
AC-T	.57	1.0	0.4	6.0	.106	0	0	.63
AE-T	.57	1.0	12.0	6.0	0	0	-.766	.40
LC-T	.57	1.0	12.0	6.0	.766	.766	0	5.00
LE-T	.57	1.0	0.4	6.0	0	-.106	-.106	.13
AC-PS	.57	1.0	0.4	6.0	.162	?	0	1.00
AE-PS	.57	1.0	10.5	6.0	0	?	-.852	-.17
LC-PS	.57	1.0	10.5	6.0	.852	?	0	6.60[3]
LE-PS	.57	1.0	0.4	6.0	0	?	-.162	.13

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
AC-T	1	85.3	.99	.66
AE-T	1	85.3	1.09	.42
LC-T	1	85.3	1.09	.42
LE-T	1	85.3	1.20	.86
AC-PS	1	85.3	1.03	.70
AE-PS	1	85.3	.97	.31
LC-PS	1	85.3	1.29	.63
LE-PS	1	85.3	1.14	.81

REMARKS:
1. Load controlled tests.
2. B assumed equal to 1 (not reported by authors).
3. Extrapolated from author's curve.

SHEAR TEST DATA SUMMARY

REFERENCE: Balasubramaniam et al. (1978) PAGE: 1/2

SOIL TESTED: Weathered Bangkok Clay

INDEX PROPERTIES

w_n:	133%	PI :	82%	ϕ_C:	22.2
w_p:	41%	σ_{vo}':		ϕ_E:	29.0
w_l:	123%	σ_{vm}':	69	G_s:	2.73

SPECIMEN PREPARATION/SAMPLING: Undisturbed piston samples

STRENGTH PROPERTIES (as reported by author)[3]

Test Type: Triaxial **Units Used by Author:** kN/m^2

TEST NO.	K	OCR	e_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIU-I	1	5.0	-	13.8	36	0	0	12.8
CIU-I	1	3.3	-	20.7	36	0	0	13.7
CIU-I	1	2.0	-	34.5	37	0	0	21.8
CIU-I	1	1.7	-	41.4	37	0	0	27.7
CIU-II	1	1.0	-	103.5	58	0	0	61.8
CIU-II	1	1.0	-	207.0	109	0	0	121.3
CIU-II	1	1.0	-	276.0	141	0	0	161.0
CIU-II	1	1.0	-	414.0	195	0	0	223.0

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
CIU-I	1	2.0	.36	.02
CIU-I	1	3.0	.38	.05
CIU-I	1	5.0	.59	.26
CIU-I	1	6.0	.75	.42
CIU-II	1	15.0	1.07	.73
CIU-II	1	30.0	1.11	.78
CIU-II	1	40.0	1.14	.81
CIU-II	1	60.0	1.14	.81

REMARKS: 1. Load controlled tests. 2. B assumed equal to 1.
3. Values of stress read from graphs, values of Δu calculated using $u_f = q_f/3 + p_o - p_f$.

SHEAR TEST DATA SUMMARY

REFERENCE: Balasubramaniam et al. (1978) PAGE: 2/2

SOIL TESTED: Weathered Bangkok Clay

INDEX PROPERTIES

w_n: 133%	PI : 82%	ϕ_C: 22.2
w_p: 41%	σ'_{vo}:	ϕ_E: 29.0
w_l: 123%	σ'_{vm}: 69	G_s: 2.73

SPECIMEN PREPARATION/SAMPLING: Undisturbed piston samples

STRENGTH PROPERTIES (as reported by author)[3]

Test Type: Triaxial **Units Used by Author:** kN/m^2

TEST NO.	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIUE-V	1	2.5	-	27.6	0	0	-26	-2.06
CIUE-V	1	2.0	-	34.5	0	0	-29	-0.17
CIUE-V	1	1.7	-	41.4	0	0	-31	3.06
CIUE-V	1	1.4	-	48.3	0	0	-44	3.63
CIUE-VIII	1	1.0	-	103.5	0	0	-81	-12.50
CIUE-VIII	1	1.0	-	207.0	0	0	-143	-6.67
CIUE-VIII	1	1.0	-	276.0	0	0	-179	5.33
CIUE-VIII	1	1.0	-	414.0	0	0	-256	25.66

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
CIU-I	1	4.0	.92	.25
CIU-I	1	5.0	.99	.33
CIU-I	1	6.0	1.10	.43
CIU-I	1	7.0	1.08	.42
CIU-II	1	15.0	.85	.18
CIU-II	1	30.0	.95	.29
CIU-II	1	40.0	1.03	.36
CIU-II	1	60.0	1.10	.43

REMARKS: 1. Load controlled tests. 2. B assumed equal to 1.
3. Values of stress read from graphs, values of u calculated using $u_f = q_f/3 + p_o - p_f$.

SHEAR TEST DATA SUMMARY

REFERENCE: Balasubramaniam et al. (1978) PAGE: 1/1

SOIL TESTED: Soft Bangkok Clay

INDEX PROPERTIES

w_n: 122%	PI : 75%	ϕ_C: 26.0
w_p: 43%	σ'_{vo}:	ϕ_E: 21.3
w_l: 118%	σ'_{vm}:	G_s: 2.75

SPECIMEN PREPARATION/SAMPLING: Undisturbed piston samples

STRENGTH PROPERTIES (as reported by author)[3]

Test Type: Triaxial Units Used by Author: kN/m^2

TEST NO.	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIU-IX	1	1.0	-	138.0	95	0	0	78.7
CIU-IX	1	1.0	-	207.0	126	0	0	133.0
CIU-IX	1	1.0	-	276.0	160	0	0	179.0
CIU-IX	1	1.0	-	345.0	225	0	0	221.0
CIUE-XIII	1	1.0	-	138.0	0	0	-118	5.67
CIUE-XIII	1	1.0	-	207.0	0	0	-129	6.00
CIUE-XIII	1	1.0	-	276.0	0	0	-177	38.00
CIUE-XIII	1	1.0	-	345.0	0	0	-196	-26.33

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
CIU-I	1	20.0	.83	.50
CIU-I	1	30.0	1.06	.72
CIU-I	1	40.0	1.12	.79
CIU-I	1	50.0	.98	.65
CIU-II	1	20.0	1.05	.38
CIU-II	1	30.0	1.05	.38
CIU-II	1	40.0	1.21	.55
CIU-II	1	50.0	.87	.20

REMARKS: 1. Load controlled tests. 2. B assumed equal to 1.
3. Values of stress read from graphs, values of Δu calculated using $u_f = q_f/3 + p_o - p_f$.

SHEAR TEST DATA SUMMARY

REFERENCE: Balasubramaniam et al. (1978) PAGE: 1/2

SOIL TESTED: Stiff Bangkok Clay

INDEX PROPERTIES

w_n: 22%	PI : 25%	ϕ_C: 24.0
w_p: 19%	σ'_{vo}: 150	ϕ_E: 25.0
w_l: 46%	σ'_{vm}:	G_s: 2.74

SPECIMEN PREPARATION/SAMPLING: Undisturbed piston samples

STRENGTH PROPERTIES (as reported by author)[3]

Test Type: Triaxial Units Used by Author: kN/m^2

TEST NO.	K	OCR	e_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIU-XVI	1	1.09	-	138.0	330	0	0	-5
CIU-XVI	1	1.00	-	207.0	353	0	0	40
CIU-XVI	1	1.00	-	276.0	368	0	0	46
CIU-XVI	1	1.00	-	345.0	454	0	0	65
CIU-XVI	1	1.00	-	414.0	477	0	0	167
CIU-XVI	1	1.00	-	483.0	567	0	0	260
CIU-XVI	1	1.00	-	552.0	598	0	0	240
CIU-XVI	1	1.00	-	620.0	679	0	0	185

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CIU-XVI	1	20.0	-.02	-.35
CIU-XVI	1	30.0	.11	-.22
CIU-XVI	1	40.0	.13	-.21
CIU-XVI	1	50.0	.14	-.19
CIU-XVI	1	60.0	.35	.02
CIU-XVI	1	70.0	.46	.13
CIU-XVI	1	80.0	.40	.07
CIU-XVI	1	90.0	.27	-.06

REMARKS: 1. Load controlled tests. 2. B assumed equal to 1.
3. Values of stress read from graphs, values of u calculated using $u_f = q_f/3 + p_o - p_f$.

SHEAR TEST DATA SUMMARY

REFERENCE: Balasubramaniam et al. (1978) PAGE: 2/2

SOIL TESTED: Stiff Bangkok Clay

INDEX PROPERTIES

w_n:	22%	PI :	25%	ϕ_C:	24.0
w_p:	19%	σ'_{vo}:	150	ϕ_E:	25.0
w_l:	46%	σ'_{vm}:		G_s:	2.74

SPECIMEN PREPARATION/[illegible]PLING: Undisturbed piston samples

STRENGTH PROPERTIES (as reported by author)[3]

Test Type: Triaxial Units Used by Author: kN/m²

TEST NO.	K	OCR	e_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
C[illegible]VIII	1	1.09	-	138.0	0	0	-260	-187
CI[illegible]XVIII	1	1.00	-	207.0	0	0	-188	-116
CIUE-XVIII	1	1.00	-	276.0	0	0	-263	-132
CIUE-XVIII	1	1.00	-	345.0	0	0	-275	-101
CIUE[illegible]II	1	1.00	-	414.0	0	0	-271	-121
CIU[illegible]II	1	1.00	-	483.0	0	0	-400	-165
[illegible]II	1	1.00	-	552.0	0	0	-400	-181
[illegible]	1	1.00	-	620.0	0	0	-432	-138

P[illegible]URE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CIU-XVIII	1	20.0	.28	-.39
CIUE-XVIII	1	30.0	.38	-.28
CIUE-XVIII	1	40.0	.50	-.17
CIUE-XVIII	1	50.0	.63	-.03
CIUE-XVIII	1	60.0	.55	-.11
CIUE-XVIII	1	70.0	.59	-.08
CIUE-XVIII	1	80.0	.55	-.12
CIUE-XVIII	1	90.0	.68	.01

REMARKS: 1. Load controlled tests. 2. B assumed equal to 1.
3. Values of stress read from graphs, values of -u calculated using $u_f = q_f/3 + p_o - p_f$.

SHEAR TEST DATA SUMMARY

REFERENCE: Paster (1982) PAGE: 1/3

SOIL TESTED: Kaolinite

INDEX PROPERTIES

w_n:	PI : 34%	ϕ_C: 20.4
w_p: 44%	σ'_{vo}:	ϕ_E: 22.8
w_l: 78%	σ'_{vm}:	G_s: 2.61

SPECIMEN PREPARATION/SAMPLING: Consolidated Slurry

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial Units Used by Author: psi

TEST NO.	K	OCR	ϵ_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIU-LD-C-1P	1	1.0	1.2	29.1	18.94	0	0	10.42
CIU-LD-C-2P	1	1.0	1.4	29.4	15.32	0	0	12.87
CIU-LD-C-3P	1	1.0	1.4	29.3	14.12	0	0	15.96
CIU-LD-C-4P	1	1.0	1.8	28.9	14.07	0	0	16.04
CIU-LD-C-5P	1	1.0	2.0	30.8	14.46	0	0	17.33
CIU-LD-C-6P	1	1.0	2.0	30.3	14.03	0	0	17.96
CIU-LD-C-7P	1	1.0	1.4	30.3	13.15	0	0	16.57

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CIU-LD-C-1P	1	29.1	.55	.22
CIU-LD-C-2P	1	29.4	.84	.51
CIU-LD-C-3P	1	29.3	1.13	.80
CIU-LD-C-4P	1	28.9	1.14	.81
CIU-LD-C-5P	1	30.8	1.20	.87
CIU-LD-C-6P	1	30.3	1.28	.95
CIU-LD-C-7P	1	30.3	1.26	.93

REMARKS: 1. Load controlled tests.

SHEAR TEST DATA SUMMARY

REFERENCE: Paster (1982) PAGE: 2/3

SOIL TESTED: Kaolinite

INDEX PROPERTIES

w_n:	PI : 34%	ϕ_C: 20.4
w_p: 44%	σ'_{vo}:	ϕ_E: 22.8
w_l: 78%	σ'_{vm}:	G_s: 2.61

SPECIMEN PREPARATION/SAMPLING: Consolidated Slurry

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial Units Used by Author: psi

TEST NO.	K	OCR	ϵ_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIU-DF-C-1P	1	1.0	1.0	30.3	14.30	0	0	14.59
CIU-DF-C-2P	1	1.0	2.0	29.5	13.63	0	0	18.54
CIU-DF-C-3P	1	1.0	2.0	30.5	13.66	0	0	19.40
CIU-DF-C-4P	1	1.0	2.0	30.5	13.48	0	0	19.55
CIU-7D-E-1P	1	1.0	0.6	30.2	0	0	-17.76	-8.43
CIU-LD-E-2P	1	1.0	2.0	30.4	0	0	-12.86	5.72
CIU-LD-E-3P	1	1.0	2.0	29.5	0	0	-12.33	5.55

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CIU-DF-C-1P	1	30.3	1.02	.69
CIU-DF-C-2P	1	29.5	1.36	1.03
CIU-DF-C-3P	1	30.5	1.42	1.09
CIU-DF-C-4P	1	30.5	1.45	1.12
CIU-LD-E-1P	1	30.2	.53	-.14
CIU-LD-E-2P	1	30.4	1.44	.78
CIU-LD-E-3P	1	29.5	1.45	.78

REMARKS: 1. Load and deformation controlled tests.

SHEAR TEST DATA SUMMARY

REFERENCE: Paster (1982) PAGE: 3/3

SOIL TESTED: Kaolinite

INDEX PROPERTIES

w_n:	PI : 34%	ϕ_C: 20.4
w_p: 44%	σ'_{vo}:	ϕ_E: 22.8
w_l: 78%	σ'_{vm}:	G_s: 2.61

SPECIMEN PREPARATION/SAMPLING: Consolidated Slurry

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial Units Used by Author: psi

TEST NO.	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CAU-LD-C-1P	.60	1.0	0.25	29.5	5.90	0	0	4.01
CAU-LD-C-2P	.60	1.0	0.25	29.8	5.04	0	0	4.45
CAU-LD-E-1P	.60	1.0	10.0	30.7	0	0	-22.13	2.76
CAU-LD-E-2P	.60	1.0	10.0	29.1	0	0	-20.17	3.35
CAU-LD-E-3P	.60	1.0	10.0	29.7	0	0	-21.32	3.30

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
CAU-LD-C-1P	1	29.5	.68	.35
CAU-LD-C-2P	1	29.8	.88	.50
CAU-LD-E-1P	1	30.7	1.12	.46
CAU-LD-E-2P	1	29.1	1.17	.50
CAU-LD-E-3P	1	29.7	1.15	.49

REMARKS: 1. Load controlled tests.

SHEAR TEST DATA SUMMARY

REFERENCE: Ladd, Bovee, Edgers, and Rixner (1971) PAGE: 1/2

SOIL TESTED: Boston Blue Clay

INDEX PROPERTIES

w_n: 36%	PI : 21%	ϕ_C:
w_p: 20%	σ'_{vo}:	ϕ_E:
w_l: 41%	σ'_{vm}:	G_s: 2.79

SPECIMEN PREPARATION/SAMPLING: Consolidated Slurry

STRENGTH PROPERTIES (as reported by author)

Test Type: Plane Strain Units Used by Author: kg/cm^2

TEST NO.	K	OCR	ϵ_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
PSA-3	.51	1.00	1.7	3.82	0.85	0.71	0.00	.860
PSA-4	.47	1.00	0.5	3.88	0.65	0.39	0.00	.430
PSA-5	.89	3.94	1.8	0.73	1.23	0.30	0.00	.100
PSA-6	.55	1.00	0.5	3.80	0.88	0.83	0.00	.790
PSA-7	.89	4.10	1.7	1.46	2.71	0.82	0.00	.495
PSA-8	.71	2.02	1.2	1.96	1.68	0.62	0.00	.570
PSA-10	.51	1.00	0.3	3.96	0.60	0.53	0.00	.520

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
PSA-3	.55	54.3	1.01	.43
PSA-4	.72	55.1	.66	.15
PSA-5	.94	10.4	.08	-.37
PSA-6	.91	54.0	.95	.25
PSA-7	.74	20.8	.18	-.28
PSA-8	.95	27.8	.34	-.13
PSA-10	1.00	56.3	.87	.25

REMARKS: 1. PSA - σ_v increased, $\Delta\sigma_h$ = 0.

SHEAR TEST DATA SUMMARY

REFERENCE: Ladd, Bovee, Edgers, and Rixner (1971) PAGE: 2/2

SOIL TESTED: Boston Blue Clay

INDEX PROPERTIES

w_n: 36%	PI : 21%	ϕ_C:
w_p: 20%	σ_{vo}':	ϕ_E:
w_l: 41%	σ_{vm}':	G_s: 2.79

SPECIMEN PREPARATION/SAMPLING: Consolidated Slurry

STRENGTH PROPERTIES (as reported by author)

Test Type: Plane Strain — Units Used by Author: kg/cm^2

TEST NO.	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
PSP-2	.51	1.00	4.2	3.85	0.05	-1.20	-3.79	-0.380
PSP-4H	.52	1.00	5.8	4.03	1.74	0.53	-1.92	1.610
PSP-9H	.66	2.00	6.5	2.09	2.29	0.96	-0.04	1.640
PSP-10	.50	1.00	4.7	3.98	0.04	-0.64	-3.38	0.217
PSP-11	.52	1.00	3.5	3.97	0.02	-0.79	-3.38	0.165
PSP-12H	.84	3.96	6.5	1.01	1.41	0.44	-0.04	0.709
PSP-13	.70	2.01	3.0	1.98	0.02	-0.57	-2.04	-3.100
PSP-22	.45	1.00	2.8	4.24	0.02	-0.99	-3.81	0.080

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
PSP-2	.71	54.8	.90	.37
PSP-4H	.89	57.3	.92	.46
PSP-9H	.94	29.7	.71	.28
PSP-10	.95	56.6	1.06	.49
PSP-11	.98	56.5	1.04	.50
PSP-12H	.99	14.4	.50	.08
PSP-13	.99	28.2	.85	.30
PSP-22	1.00	60.3	1.02	49

REMARKS: 1. PSP - σ_v decreased, $\Delta\sigma_h$ = 0.
2. PSP-4H - $\sigma_{vc} = K_o\ \sigma_{hc}$, σ_v increased , $\Delta\sigma_h$ = 0.

SHEAR TEST DATA SUMMARY

REFERENCE: Fennessey (1983) PAGE: 1/5

SOIL TESTED: Gleason Clay (Commercially available)

INDEX PROPERTIES

w_n:	PI : 25%	ϕ_C: 22.7
w_p: 29%	σ_{vo}':	ϕ_E: 29.4
w_l: 54%	σ_{vm}':	G_s: 2.66

SPECIMEN PREPARATION/SAMPLING: Consolidated Slurry

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial **Units Used:** psi

TEST NO.[1]	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1/\sigma_c'$	$\Delta\sigma_2$	$\Delta\sigma_3$	$\Delta u/\sigma_c'$
CIU-LC1	1	1.0	7.6	-	.477	0	0	.622
CIU-LC2	1	1.0	9.0	-	.494	0	0	.627
CIU-LC3	1	1.0	7.2	-	.492	0	0	.602
CIU-LC4	1	1.0	5.2	-	.471	0	0	.584
CIU-LC5	1	1.0	7.4	-	.503	0	0	.610
CIU-LC6	1	1.0	6.4	-	.490	0	0	.565
CIU-DC1	1	1.0	6.2	-	.466	0	0	.597
CIU-DC2	1	1.0	12.5	-	.574	0	0	.625

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.[1]	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
CIU-LC1	1	-	1.30	.97
CIU-LC2	1	-	1.27	.94
CIU-LC3	1	-	1.22	.89
CIU-LC4	1	-	1.24	.91
CIU-LC5	1	-	1.21	.88
CIU-LC6	1	-	1.15	.82
CIU-DC1	1	-	1.28	.95
CIU-DC2	1	-	1.09	.76

REMARKS: 1. LC - load controlled, DC - deformation controlled.
2. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Fennessey (1983) PAGE: 2/5

SOIL TESTED: Gleason Clay (Commercially available)

INDEX PROPERTIES

w_n:	PI : 25%	ϕ_C: 22.7
w_p: 29%	σ'_{vo}:	ϕ_E: 29.4
w_l: 54%	σ'_{vm}:	G_s: 2.66

SPECIMEN PREPARATION/SAMPLING: Consolidated Slurry

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial Units Used: psi

TEST NO.[1]	K	OCR	ϵ_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3/\sigma'_c$	$\Delta u/\sigma'_c$
CIU-LE1	1	1.0	14.0	-	0	0	-.518	.212
CIU-LE2	1	1.0	19.0	-	0	0	-.550	.194
CIU-LE3	1	1.0	18.0	-	0	0	-.540	.183
CIU-DE1	1	1.0	13.0	-	0	0	-.525	.201
CIU-DE2	1	1.0	13.0	-	0	0	-.515	.189

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.[1]	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CIU-LE1	1	-	1.41	.74
CIU-LE2	1	-	1.35	.69
CIU-LE3	1	-	1.34	.67
CIU-DE1	1	-	1.38	.72
CIU-DE2	1	-	1.37	.70

REMARKS: 1. LE - load controlled, DE - deformation controlled.
2. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Fennessey (1983) PAGE: 3/5

SOIL TESTED: Gleason Clay (Commercially available)

INDEX PROPERTIES

w_n:	PI : 25%	ϕ_C: 22.7
w_p: 29%	σ_{vo}':	ϕ_E: 29.4
w_l: 54%	σ_{vm}':	G_s: 2.66

SPECIMEN PREPARATION/SAMPLING: Consolidated Slurry

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial Units Used: psi

TEST NO.[1]	K	OCR	e_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CAU-C9	.57	1.0	1.4	30.2	2.93	0	0	3.29
CAU-C10	.57	1.0	1.3	30.2	2.72	0	0	3.11
CAU-E2	.57	1.0	10.0	29.6	0	0	-24.10	0.03
CAU-E3	.57	1.0	20.0	30.9	0	0	-25.50	-1.58
CAU-E4	.57	1.0	20.0	29.9	0	0	-25.78	-0.95

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.[1]	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
CAU-C9	1	30.2	1.12	.79
CAU-C10	1	30.2	1.14	.81
CAU-E2	1	29.6	1.00	.33
CAU-E3	1	30.9	.94	.27
CAU-E4	1	29.9	.96	.30

REMARKS: 1. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Fennessey (1983) PAGE: 4/5

SOIL TESTED: Gleason Clay (Commercially available)

INDEX PROPERTIES

w_n:		PI :	25%	ϕ_C:	22.7
w_p:	29%	σ'_{vo}:		ϕ_E:	29.4
w_l:	54%	σ'_{vm}:		G_s:	2.66

SPECIMEN PREPARATION/SAMPLING: Consolidated Slurry

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial Units Used: psi

TEST NO.[1]	K	OCR	ϵ_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CAU-C1	.66	1.0	1.5	29.8	4.41	0	0	5.51
CAU-C2	.67	1.0	1.4	29.7	4.43	0	0	5.35
CAU-E1	.67	1.0	20.0	30.3	0	0	-23.3	0.15
CAU-E2	.66	1.0	20.0	30.4	0	0	-24.1	0.52
CAU-C2	.77	1.0	6.0	30.4	7.48	0	0	11.49
CAU-C3	.77	1.0	5.5	30.1	7.13	0	0	11.26
CAU-E1	.77	1.0	20.0	30.1	0	0	-21.8	2.44
CAU-E2	.77	1.0	20.0	30.3	0	0	-21.7	3.03

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.[1]	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CAU-C1	1	-	1.25	.92
CAU-C2	1	-	1.21	.87
CAU-E1	1	-	1.01	.34
CAU-E2	1	-	1.02	.35
CAU-C2	1	-	1.54	1.20
CAU-C3	1	-	1.58	1.24
CAU-E1	1	-	1.11	.45
CAU-E2	1	-	1.14	.47

REMARKS: 1. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Fennessey (1983) PAGE: 5/5

SOIL TESTED: Hydrite UF (Commercially available)

INDEX PROPERTIES

w_n:	PI : 34%	ϕ_C: 22.9
w_p: 44%	σ_{vo}':	ϕ_E: 36.7
w_l: 78%	σ_{vm}':	G_s: 2.61

SPECIMEN PREPARATION/SAMPLING: Consolidated Slurry

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial Units Used: psi

TEST NO.[1]	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1/\sigma_c'$	$\Delta\sigma_2$	$\Delta\sigma_3/\sigma_c'$	$\Delta u/\sigma_c'$
CAU-R-1C	.60	1.0	.24	-	.192	0	0	.138
CAU-R-2C	.60	1.0	.34	-	.191	0	0	.148
CAU-S-1C	.62	1.0	.28	-	.216	0	0	.154
CAU-S-3C	.60	1.0	.28	-	.175	0	0	.151
CAU-R-1E	.60	1.0	17.0	-	0	0	-.742	.169
CAU-R-2E	.60	1.0	19.0	-	0	0	-.751	.148
CAU-S-2E	.64	1.0	14.0	-	0	0	-.709	.124
CAU-S-3E	.61	1.0	10.0	-	0	0	-.768	.094

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.[1]	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
CAU-R-1C	1	-	.72	.39
CAU-R-2C	1	-	.77	.44
CAU-S-1C	1	-	.71	.38
CAU-S-3C	1	-	.86	.53
CAU-R-1E	1	-	1.23	.56
CAU-R-2E	1	-	1.20	.53
CAU-S-2E	1	-	1.17	.51
CAU-S-3E	1	-	1.12	.46

REMARKS: 1. R - load controlled, S - deformation controlled.
2. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Parry and Nadarajah (1973) PAGE: 1/2

SOIL TESTED: Kaolin

INDEX PROPERTIES

w_n:	PI :	ϕ_C:
w_p: 40%	σ'_{vo}:	ϕ_E:
w_l: 72%	σ'_{vm}:	G_s:

SPECIMEN PREPARATION/SAMPLING: Consolidated Slurry

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial **Units Used by Author:** kPa

TEST NO.	K	OCR	e_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIUC1	1	1.0	16.9	550	235	0	0	349
CIUC2	1	1.3	15.5	424	235	0	0	230
CIUC3	1	1.6	14.5	344	210	0	0	164
CIUC4	1	2.3	16.0	241	190	0	0	84
CIUE1	1	1.0	5.5	550	0	-229	-229	101
CIUE2	1	1.3	5.3	424	0	-214	-214	6.6
CIUE3	1	1.6	5.3	344	0	-200	-200	-38
CIUE4	1	2.3	4.4	241	0	-166	-166	-81

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CIUC1	1	80.0	1.49	1.15
CIUC2	1	61.0	.98	.65
CIUC3	1	50.0	.80	.45
CIUC4	1	35.0	.44	.11
CIUE1	1	80.0	1.44	.77
CIUE2	1	61.0	1.03	.36
CIUE3	1	50.0	.81	.14
CIUE4	1	35.0	.51	-.15

REMARKS: 1. Load controlled tests.
2. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Parry and Nadarajah (1973) PAGE: 2/2

SOIL TESTED: Kaolin

INDEX PROPERTIES

w_n: _____ PI : _____ ϕ_C: _____

w_p: 40% σ'_{vo}: _____ ϕ_E: _____

w_l: 72% σ'_{vm}: _____ G_s: _____

SPECIMEN PREPARATION/SAMPLING: Consolidated Slurry

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial **Units Used by Author:** kPa

TEST NO.	K	OCR	ϵ_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CAUC1	.64	1.0	5.2	545	42	0	0	116
CAUC2	.64	1.4	4.4	379	167	0	0	99
CAUC3	.64	2.0	6.0	274	189	0	0	60
CAUC4	.64	2.6	4.0	210	176	0	0	44
CAUE1	.64	1.0	18.8	540	0	-386	-386	44
CAUE2	.64	1.4	17.0	376	0	-258	-258	-19
CAUE3	.64	2.0	15.7	276	0	-189	-189	-11
CAUE4	.64	2.6	17.2	218	0	-166	-166	-48

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CAUC1	1	79.0	2.76	2.43
CAUC2	1	55.0	.59	.26
CAUC3	1	40.0	.32	-.02
CAUC4	1	30.0	.25	-.08
CAUE1	1	79.0	1.41	.45
CAUE2	1	55.0	1.07	.41
CAUE3	1	40.0	.94	.28
CAUE4	1	30.0	.71	.04

REMARKS: 1. Load controlled tests.
2. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Parry (1960) PAGE: 1/5

SOIL TESTED: Weald Clay

INDEX PROPERTIES

w_n:	PI : 25%	ϕ_C: 19.8
w_p: 18%	σ_{vo}':	ϕ_E: 16.0
w_l: 43%	σ_{vm}':	G_s:

SPECIMEN PREPARATION/SAMPLING: Remolded

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial **Units Used:** psi

TEST NO.	K	OCR	e_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIU-AC1	1	1.0	20.0	15	9.9	0	0	7.3
CIU-AC2	1	1.3	17.5	30	17.3	0	0	16.5
CIU-AC3	1	1.6	15.0	30	25.0	0	0	24.9
CIU-AC4	1	2.3	13.5	60	32.6	0	0	33.0
CIU-AC5	1	1.0	12.0	120	68.0	0	0	66.5
CIU-AE1	1	1.3	7.0	30	0	0	-14.2	4.0
CIU-AE2	1	1.6	8.0	60	0	0	-28.3	7.8
CIU-AE3	1	2.3	7.5	120	0	0	-55.7	16.0

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
CIU-AC1	1	15.0	0.74	0.40
CIU-AC2	1	30.0	0.95	0.62
CIU-AC3	1	30.0	1.00	0.66
CIU-AC4	1	60.0	1.01	0.68
CIU-AC5	1	120.0	0.98	0.64
CIU-AE1	1	30.0	1.28	0.62
CIU-AE2	1	60.0	1.28	0.61
CIU-AE3	1	120.0	1.29	0.62

REMARKS: 1. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Parry (1960) PAGE: 2/5

SOIL TESTED: Weald Clay

INDEX PROPERTIES

w_n: ______	PI : 25%	ϕ_C: 19.8
w_p: 18%	σ'_{vo}: ______	ϕ_E: 16.0
w_l: 43%	σ'_{vm}: ______	G_s: ______

SPECIMEN PREPARATION/SAMPLING: Remolded

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial **Units Used:** psi

TEST NO.	K	OCR	e_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIU-AC6	1	2.0	17.5	15	13.3	0	0	4.2
CIU-AC7	1	2.1	13.5	30	29.0	0	0	8.4
CIU-AC8	1	2.7	17.5	22	23.7	0	0	3.6
CIU-AC9	1	1.7	12.0	70	58.6	0	0	21.6
CIU-AC10	1	2.0	13.5	60	54.7	0	0	14.5
CIU-AE4	1	2.0	7.5	30	0	0	-23.0	-12.0
CIU-AE5	1	1.7	7.0	70	0	0	-49.1	-18.4
CIU-AE6	1	2.0	7.5	60	0	0	-45.2	-23.7

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CIU-AC6	1	15.0	0.32	-0.02
CIU-AC7	1	30.0	0.29	-0.04
CIU-AC8	1	22.0	0.15	-0.18
CIU-AC9	1	70.5	0.37	0.04
CIU-AC10	1	60.0	0.27	-0.07
CIU-AE4	1	30.0	0.48	-0.19
CIU-AE5	1	70.5	0.63	-0.04
CIU-AE6	1	60.0	0.48	-0.19

REMARKS: 1. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Parry (1960) PAGE: 3/5

SOIL TESTED: Weald Clay

INDEX PROPERTIES

w_n:		PI :	25%	ϕ_C:	19.8
w_p:	18%	σ_{vo}':		ϕ_E:	16.0
w_l:	43%	σ_{vm}':		G_s:	

SPECIMEN PREPARATION/SAMPLING: Remolded

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial Units Used: psi

TEST NO.	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIU-AC11	1	4.0	21.0	7.5	10.1	0	0	-1.0
CIU-AC12	1	4.0	16.0	15.0	20.0	0	0	0.1
CIU-AC13	1	4.0	15.0	30.0	39.2	0	0	-0.7
CIU-AC14	1	8.0	16.5	15.0	27.8	0	0	-2.7
CIU-AE7	1	4.0	8.0	15.0	0	0	-15.8	-14.2
CIU-AE8	1	4.0	7.0	30.0	0	0	-34.4	-28.2
CIU-AE9	1	8.0	8.0	15.0	0	0	-22.4	-22.5

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
CIU-AC11	1	7.5	-0.10	-0.43
CIU-AC12	1	15.0	0.01	-0.33
CIU-AC13	1	30.0	-0.02	-0.35
CIU-AC14	1	15.0	-0.10	-0.43
CIU-AE7	1	15.0	0.10	-0.57
CIU-AE8	1	30.0	0.18	-0.49
CIU-AE9	1	15.0	-0.01	-0.67

REMARKS: 1. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Parry (1960) PAGE: 4/5

SOIL TESTED: Weald Clay

INDEX PROPERTIES

w_n:	PI : 25%	ϕ_C: 19.8
w_p: 18%	σ'_{vo}:	ϕ_E: 16.0
w_l: 43%	σ'_{vm}:	G_s:

SPECIMEN PREPARATION/SAMPLING: Remolded

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial **Units Used:** psi

TEST NO.	K	OCR	e_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIU-AC15	1	12.0	21.0	5	11.3	0	0	-3.0
CIU-AC16	1	12.0	16.5	10	20.0	0	0	-6.0
CIU-AC17	1	24.5	17.5	4.9	13.4	0	0	-4.8
CIU-AE10	1	12.0	10.0	5	0		-10.1	-11.5
CIU-AE11	1	12.0	10.5	10	68.0	0	-18.9	-19.8
CIU-AE12	1	24.0	10.0	5	0	0	-12.5	-14.1

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CIU-AC15	1	5.0	-0.27	-0.60
CIU-AC16	1	10.0	-0.30	-0.63
CIU-AC17	1	4.9	-0.34	-0.68
CIU-AE10	1	5.0	-0.14	-0.81
CIU-AE11	1	10.0	-0.05	-0.71
CIU-AE12	1	5.0	-0.13	-0.79

REMARKS: 1. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Parry (1960) PAGE: 5/5

SOIL TESTED: Weald Clay

INDEX PROPERTIES

w_n: ____	PI : 25%	ϕ_C: 19.8
w_p: 18%	σ_{vo}': ____	ϕ_E: 16.0
w_l: 43%	σ_{vm}': ____	G_s: ____

SPECIMEN PREPARATION/SAMPLING: Remolded

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial **Units Used:** psi

TEST NO.	K	OCR	e_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIU-LE1	1	1.0	15.0	30	0	-15.4	-15.4	-0.9
CIU-LE2	1	4.0	13.5	30	0	-38.7	-38.7	40.9
CIU-LC1	1	1.0	7.0	30	14.8	14.8	0	18.7
CIU-LC2	1	4.0	8.0	15	33.5	33.5	0	3.6
CIU-LC3	1	8.0	8.5	15	23.7	23.7	0	-1.8
CIU-LC4	1	12.0	9.0	5	10.3	10.3	0	-1.6
CIU-LC5	1	12.0	8.5	10	18.8	18.8	0	-1.9
CIU-LC6	1	24.0	10.0	5	11.6	11.6	0	-1.8

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
CIU-LE1	1	30.0	0.94	0.61
CIU-LE2	1	30.0	2.06	1.72
CIU-LC1	1	30.0	1.26	0.60
CIU-LC2	1	15.0	0.11	-.56
CIU-LC3	1	15.0	-.08	-.74
CIU-LC4	1	5.0	-.16	-.82
CIU-LC5	1	10.0	-.10	-.77
CIU-LC6	1	5.0	-.16	-.82

REMARKS: 1. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Badillo E. J. (1964) PAGE: 1/2

SOIL TESTED: Weald Clay

INDEX PROPERTIES

w_n:	34%	PI :	25%	ϕ_C:	19.8
w_p:	18%	σ_{vo}':		ϕ_E:	16.0
w_l:	43%	σ_{vm}':		G_s:	

SPECIMEN PREPARATION/SAMPLING: Remolded

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial **Units Used:** psi

TEST NO.	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIU-AC1	1	1.0	-	120	69.0	0	0	68.8
CIU-AC2	1	1.7	-	70	56.0	0	0	24.8
CIU-AC3	1	2.0	-	60	52.8	0	0	17.7
CIU-AC4	1	4.0	-	30	39.0	0	0	-0.9
CIU-AC5	1	8.0	-	15	27.8	0	0	-6.0
CIU-AC6	1	12.0	-	10	22.5	0	0	-6.0
CIU-AC7	1	24.0	-	5	15.0	0	0	-5.3

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
CIU-AC1	1	120.0	0.94	0.61
CIU-AC2	1	70.0	0.44	0.11
CIU-AC3	1	60.0	0.34	0.00
CIU-AC4	1	30.0	-.02	-.36
CIU-AC5	1	15.0	-.22	-.55
CIU-AC6	1	10.0	-.27	-.60
CIU-AC7	1	5.0	-.35	-.68

REMARKS: 1. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Badillo E. J. (1964)

SOIL TESTED: Weald Clay

INDEX PROPERTIES

w_n: 34%	PI : 25%	ϕ_C: 19.8
w_p: 18%	σ'_{vo}:	ϕ_E: 16.0
w_l: 43%	σ'_{vm}:	G_s:

SPECIMEN PREPARATION/SAMPLING: Remolded

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial **Units Used:** psi

TEST NO.	K	OCR	ϵ_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIU-AE1	1	1.0	-	120	0	0	-56.4	15.6
CIU-AE2	1	1.7	-	70	0	0	-49.0	-18.2
CIU-AE3	1	2.0	-	60	0	0	-45.6	-24.0
CIU-AE4	1	4.0	-	30	0	0	-33.6	-28.8
CIU-AE5	1	8.0	-	15	0	0	-23.3	-24.8
CIU-AE6	1	12.0	-	10	0	0	-11.0	-21.5
CIU-AE7	1	24.0	-	5	0	0	-12.5	-14.0

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CIU-AE1	1	120.0	1.28	0.61
CIU-AE2	1	70.0	0.63	-.04
CIU-AE3	1	60.0	0.47	-.19
CIU-AE4	1	30.0	0.14	-.52
CIU-AE5	1	15.0	-.06	-.73
CIU-AE6	1	10.0	-.13	-.80
CIU-AE7	1	5.0	-.12	-.79

REMARKS: 1. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Wu, Loh, and Malvern (1963) PAGE: 1/1

SOIL TESTED: Sault Saint Marie Clay

INDEX PROPERTIES

w_n:	PI : 24%	ϕ_C: 32
w_p: 28%	σ'_{vo}:	ϕ_E:
w_l: 52%	σ'_{vm}:	G_s:

SPECIMEN PREPARATION/SAMPLING: Remolded

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial **Units Used:** kg/cm^2

TEST NO.	K	OCR	ϵ_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIUC1	1	1.0	0.15	1.00	1.10	0	0	0.58
CIUC2	1	1.0	0.15	4.00	3.85	0	0	2.40
CIUC3	1	1.0	0.15	2.00	2.25	0	0	1.08
CIUE1	1	1.0	0.10	3.81	0	0	-2.53	0.21
CIUE2	1	1.0	0.11	2.85	0	0	-1.83	0.27
CIUE3	1	1.0	0.08	1.89	0	0	-1.28	0.09

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CIUC1	1	14.2	0.53	0.19
CIUC2	1	56.9	0.62	0.29
CIUC3	1	28.4	0.48	0.15
CIUE1	1	54.2	1.08	0.42
CIUE2	1	40.5	1.15	0.48
CIUE3	1	26.9	1.07	0.40

REMARKS: 1. B assumed equal to 1.

SHEAR TEST DATA SUMMARY

REFERENCE: Ladd and Edgers (1972) PAGE: 1/3

SOIL TESTED: Portsmouth Sensitive Marine Clay

INDEX PROPERTIES

w_n:	50%	PI :	15%	ϕ_C:	21
w_p:	20%	σ'_{vo}:	.50	ϕ_E:	
w_l:	35%	σ'_{vm}:	.84	G_s:	2.74

SPECIMEN PREPARATION/SAMPLING: Undisturbed/ 5" fixed piston

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial-Plane Strain **Units Used by Author:** kg/cm^2

TEST NO.	K	OCR	e_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIUC-1	1	1.0	1.3	1.0	.295	0	0	-
CIUC-2	1	1.0	2.0	2.0	.290	0	0	-
CIUC-3	1	1.0	2.6	1.0	.270	0	0	-
CIUC-5	1	1.0	1.6	1.1	.257	0	0	-
CIUC-6	1	1.0	2.1	2.2	.245	0	0	-
CAUC-1	.625	1.0	0.5	1.4	.271	0	0	-
CK_oUPSA	.475	1.0	0.3	1.4	.350	0	0	-
CK_oUPSP	.450	1.0	5.9	2.4	0	?	-.155	-

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
CIUC-1	1	14.2	.73	.40
CIUC-2	1	28.4	.86	.53
CIUC-3	1	14.2	.89	.56
CIUC-5	1	14.2	1.05	.72
CIUC-6	1	31.2	1.23	.90
CAUC-1	1	19.9	1.48	1.150
CK_oUPSA	1	19.9	.94	.61
CK_oUPSP	1	34.1	1.01	.35

REMARKS: 1. B assumed equal to 1 (not reported by author).

SHEAR TEST DATA SUMMARY

REFERENCE: Ladd and Edgers (1972) PAGE: 2/3

SOIL TESTED: Atchafalaya Clay

INDEX PROPERTIES

w_n:	95%	PI :	75%	ϕ_C:	21
w_p:	20%	σ_{vo}':	1200	ϕ_E:	24
w_l:	95%	σ_{vm}':		G_s:	

SPECIMEN PREPARATION/SAMPLING: Undisturbed/ 5" fixed piston

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial-Plane Strain **Units Used by Author:** kg/cm^2

TEST NO.	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
CIUC-1	1	1.0	6.7	2.5	.261	0	0	-
$CK_oUP\check{S}A$	.68	1.0	0.7	1.3	.310	0	0	-
CK_oUC	.30	1.0	3.0	1.7	.240	0	0	-
CK_oUE	.69	1.0	15.0	1.6	0	0	.540	-

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
CIUC-1	1	35.5	1.16	.83
CK_oUPSA	1	18.5	.80	.47
CK_oUC	1	24.1	1.40	1.07
CK_oUE	1	22.7	.90	.24

REMARKS: 1. Strain controlled tests.
2. B assumed equal to 1 (not reported by authors).

SHEAR TEST DATA SUMMARY

REFERENCE: Ladd and Edgers (1972) PAGE: 3/3

SOIL TESTED: Connecticut Valley Varved Clay

INDEX PROPERTIES

w_n:		PI :	39%	ϕ_C:	
w_p:	26%	σ_{vo}':		ϕ_E:	
w_l:	65%	σ_{vm}':		G_s:	

SPECIMEN PREPARATION/SAMPLING: Undisturbed/ 5" fixed piston

STRENGTH PROPERTIES (as reported by author)

Test Type: Triaxial-Plane Strain **Units Used by Author:** kg/cm^2

TEST NO.	K	OCR	ϵ_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
PSA-1	.67	1.0	.4	4.01	.279	0	0	-
PSP-1	.67	1.0	4.4	4.01	0	0	-.262	-
PSP-2	.68	1.0	4.0	4.00	0	0	-.251	-
C-1	.68	1.0	1.0	3.41	.252	0	0	-
C-4	.68	1.0	0.8	4.00	.234	0	0	-
E-1	.68	1.0	6.2	3.44	0	0	-.142	-
E-2	.73	1.0	9.1	4.04	0	0	-.162	-

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
PSA-1	1	56.8	1.09	.73
PSP-1	1	56.8	.78	.12
PSP-2	1	56.8	.86	.20
C-1	1	48.4	1.32	.99
C-4	1	56.8	1.42	1.09
E-1	1	48.8	1.24	.58
E-2	1	56.8	1.19	.53

REMARKS: 1. B assumed equal to 1 (not reported by author).

SHEAR TEST DATA SUMMARY

REFERENCE: Duncan and Seed (1966) PAGE: 1/2

SOIL TESTED: San Francisco Bay Mud

INDEX PROPERTIES

w_n: 90%	PI : 45%	ϕ_C: 38
w_p: 43%	σ_{vo}':	ϕ_E: 35
w_l: 88%	σ_{vm}':	G_s:

SPECIMEN PREPARATION/SAMPLING: Undisturbed/ fixed piston

STRENGTH PROPERTIES (as reported by author)

Test Type: Plane Strain Units Used by Author: kg/cm^2

TEST NO.	K	OCR	e_f	σ_c'	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
VPS-3	.49	1.0	3.1	3.67	.84	0	0	.90
VPS-4	.55	1.0	4.0	0.64	.22	0	0	.20
VPS-5	.50	1.0	3.2	1.39	.34	0	0	.39
VPS-6	.50	1.0	4.2	2.14	.51	0	0	.58
VPS-8	.50	1.0	3.8	2.90	.66	0	0	.79
VPS-9	.48	1.0	3.5	3.72	.81	0	0	.87

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ_c'(psi)	A_f	$a_f\sqrt{2}$
VPS-3	1	52.0	1.07	.74
VPS-4	1	9.1	.91	.58
VPS-5	1	19.7	1.15	.81
VPS-6	1	30.4	1.14	.80
VPS-8	1	41.2	1.20	.86
VPS-9	1	52.8	1.07	.74

REMARKS: 1. B assumed equal to 1 (not reported by authors).

AD-A157 910 AN EVALUATION OF THE USE OF PORE PRESSURE COEFFICIENTS IN TNE ANALYSIS OF..(U) AIR FORCE INST OF TECH WRIGHT-PATTERSON AFB OH R VAN SAUN AUG 85 3/3

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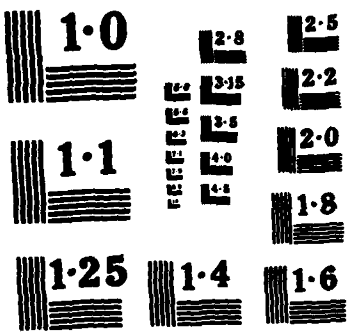

NATIONAL BUREAU OF STANDARDS
MICROCOPY RESOLUTION TEST CHART

SHEAR TEST DATA SUMMARY

REFERENCE: Duncan and Seed (1966) PAGE: 2/2

SOIL TESTED: San Francisco Bay Mud

INDEX PROPERTIES

w_n: 90% PI : 45% ϕ_C: 38
w_p: 43% σ'_{vo}: ______ ϕ_E: 35
w_l: 88% σ'_{vm}: ______ G_s: ______

SPECIMEN PREPARATION/SAMPLING: Undisturbed/ fixed piston

STRENGTH PROPERTIES (as reported by author)

Test Type: Plane Strain **Units Used by Author:** kg/cm^2

TEST NO.	K	OCR	θ_f	σ'_c	$\Delta\sigma_1$	$\Delta\sigma_2$	$\Delta\sigma_3$	Δu
HPS-4	.43	1.0	11.1	0.69	.81	0	-.92	.38
HPS-5	.38	1.0	9.6	1.20	1.65	0	-1.95	.57
HPS-6	.42	1.0	9.0	0.34	0.41	0	-0.47	.14
HPS-7	.41	1.0	10.0	0.99	1.28	0	-1.41	.55
HPS-9	.37	1.0	10.6	1.47	2.23	0	-2.57	.66
HPS-11	.42	1.0	10.6	1.66	2.24	0	-2.34	.86
HPS-12	.48	1.0	9.8	1.52	1.77	0	-1.66	86

PORE PRESSURE COEFFICIENT PARAMETERS

TEST NO.	B	σ'_c(psi)	A_f	$a_f\sqrt{2}$
HPS-4	1	9.8	.75	.08
HPS-5	1	17.0	.70	.03
HPS-6	1	4.8	.69	.03
HPS-7	1	14.0	.73	.06
HPS-9	1	20.9	.67	.00
HPS-11	1	23.5	.70	.03
HPS-12	1	21.6	.73	.06

REMARKS: 1. B assumed equal to 1 (not reported by authors).

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